December 2012

Assessing the Feasibility of Increasing Water Capacity Between the Paraíso Pumping Station and the Miraflores Potable Water Plant

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A Major Qualifying Project

submitted to the Faculty of

Worcester Polytechnic Institute

in partial fulfillment of the requirements for the

Degree of Bachelor of Science

In cooperation with

The Panama Canal Authority

Submitted on January 8th, 2013.

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Abstract

This project evaluated the feasibility of different proposed solutions to increase water capacity between the Paraíso Raw Water Pumping Station and the Miraflores Potable Water Plant, including installing a third raw water line from the Paraíso Pumping Station to the Miraflores Potable Water Plant and utilizing a rainwater catchment at the Pedro Miguel sub-basin. All facilities under consideration in the study are currently owned and operated by the Panama Canal Authority (ACP). The current water demand from this system is 50 MGD. In order to produce this water demand, the pumps at the Paraíso Pumping Station must overcome a significant friction head within the lines. This requires a substantial amount of energy and is very costly to the ACP. A third water line is proposed to reduce energy needs and operating costs at the current water demand while also providing room for future growth. A gravity fed rainwater catchment from the Pedro Miguel sub-basin would reduce pumping energy for those given flows. Additional analysis was performed for a projected demand of 70 MGD at the Miraflores Plant, requiring added flow from the Pumping Station to the Plant. Considerations of the pipeline capacities, pump demands, total energy reduction, and construction costs were used in the feasibility study. Total costs for each alternative were determined in order to compare the most cost efficient and desirable alternative based on their internal rate of return and actual net value against a base alternative. The results from this study provide the ACP with recommendations about which alternative to pursue given the current and future demands.

Executive Summary

The Panama Canal has been a vital resource to the global maritime market since it opened in 1914 (Panama Canal Authority, 2012a). An endeavor initially started by the French in 1876 as a sea-level canal, the United States took over the construction in 1905 and worked on building a canal with a system of locks (Panama Canal Authority, 2012b; Panama Canal Authority, 2012c). The completion of the Canal allowed for ships, up to a certain size, to traverse through the Canal instead of travelling around South America to travel between the Atlantic Ocean and the Pacific Ocean.

Due to the rocky terrain that makes up the Isthmus of Panama, a sea level canal proved to be challenging to excavate, leading to the ultimate demise of the French efforts. To reduce the amount of rock that would need to be removed, the Americans devised a lock system. An earthen dam was created to flood an area in the middle of the isthmus, creating Gatun Lake (Panama Canal Authority, 2012d). Gatun Lake is approximately 85 feet above sea level. Ships must travel through one set of locks on the Atlantic side known as the Gatun Locks, traversing through three chambers before reaching the elevation of Gatun Lake (Panama Canal Authority, 2012e). Ships must travel through two sets of lock on the Pacific side. Two chambers make up the Miraflores Locks and raise ships from the Pacific Ocean to Miraflores Lake. One lock chamber makes up the Pedro Miguel Locks and raises ships from the Miraflores Lake to Gatun Lake. Each lock features parallel lock chambers so that two ships may traverse a lock at a time.

Currently, the Panama Canal Authority (ACP) is undertaking a project to expand the Panama Canal so that there will be three lanes of travel for ships. The new third lane will feature larger lock chambers so that ships that are currently too big to fit in the locks may traverse the Canal. In addition to this large scale project, the ACP undertakes many other projects to improve the Canal, improve the ACP operations and improve the areas of Greater Panama.

One such project is focused on reducing the energy usage at a raw water pumping station. Currently, the ACP produces the energy that they use to operate their facilities. Any excess energy that is generated and not used is then sold to the electrical utility. The project in question aims to reduce the energy used at the pumping station, which will allow the system to operate more efficiently, but it will also provide the opportunity for the ACP to sell the saved energy for a profit.

Project Objectives

The goal of this project was to assess the feasibility of increasing water capacity between the Paraíso Pumping Station and the Miraflores Potable Water Plant. Two major options were considered. The first option that was considered was the addition of a third pipeline to run between these two facilities. The second option that was considered was a connection to the Pedro Miguel Rainwater Catchment line to a third pipeline. In order to determine the feasibility of each alternative, the following information was determined:

The flows within each pipeline: a model was created in Excel to calculate the flows within each pipeline based on the total flow demand to the Miraflores Potable Water Plant, the length and diameter of each pipe, and the pipe material. The flows were adjusted to ensure that the friction head losses in each pipe were roughly equivalent.

Energy that is used by the pumps at the Paraíso Pumping Station: based off of the flows that were calculated and the subsequent head loss in each pipeline, the pump head was calculated. The pump head was used to calculate energy used. The energy per pipeline could then be summed for all of the pumps in the system.

The cost of procurement of materials as well as construction and installation costs: in order to determine whether each alternative was financially feasible long term, the cost of materials and installation were calculated considering a 60% installation cost and a 25% contingency cost. This cost was then compared to the energy savings to determine at what point in the future, the project would be paid off.

This project also created a preliminary design for the proposed third pipeline, the rainwater catchment and the junction that occurs between the two lines. The design of the pipeline sought to find the shortest path between the Paraíso Pumping Station and the Miraflores Potable Water Plant that minimized bends in the pipe, road crossings and railroad crossings. The junction between the third pipeline and the rainwater catchment pipeline was designed with a junction angle that would reduce the head loss at the junction.

Project Outcomes

It was determined that for the current scenario, it costs approximately $1.56 million to meet the 11.2 million kWh that it takes to operate the pumps. The efficiency of the Paraíso Pumping Station is 69.2 %. The annual operating costs for the different alternatives were calculated and ranged between approximately $200,000 and $1,700,000. The procurement and installation costs for different sized pipelines ranged between approximately $1.4 million and $6.2 million. The procurement and installation costs were compared to the annual operating costs using an economic analysis spreadsheet that was provided by the Panama Canal Authority. This spreadsheet yielded the annual net value of each alternative and the internal rate of each alternative.

The optimal design of the third pipeline yielded a length of approximately 4320 meters. This path also minimized the bends in the pipeline, the road crossings and the railroad crossings. The junction angle that yielded the smallest head loss was calculated to be 165 degrees.

Project Conclusions and Recommendations

Based on the current flow demands from the Miraflores Potable Water Plant, it is recommended that a third fiberglass line is installed with a 36 inch diameter. This action will reduce the friction head loss in the pipelines which will reduce the need for pumping. For the time being, the energy savings from the Pumping Station can be sold by the ACP for profit. The installation of the 36 inch third pipeline will also prepare the Pumping Station to be able to handle increased flows that are anticipated in the future.

For the projected future flow demands from the Miraflores Potable Water Plant, it is recommended that a rainwater catchment pipeline be installed in the Pedro Miguel River sub-basin. This pipeline can be connected to the third pipeline and will supplement the flow within that line.

Capstone Design Criteria

The project was completed in order to fulfill ABET's Capstone design Criteria needed for the successful completion of a Bachelor of Science degree in engineering for the both of the project authors. The project involved several consideration including economic, environmental, sustainability, health and safety, and political factors of the work performed.

Acknowledgements

We would like to thank the individuals and organizations that provided support throughout the duration of this project.

First, we would like to thank our sponsor supervisor, Mr. Urho Gonzal, from the Panama Canal Authority. Mr. Gonzal worked with our team consistently for the duration of the project and provided us with many resources that were vital to our study.

We would also like to thank Roy Phillips of the Panama Canal Authority for providing us with assistance, a welcoming personality and for bringing us to Colon to view the current expansion of the Canal.

We would also like to thank Emilio Messina of the Panama Canal Authority for providing us with several resources about the watershed and for arranging a tour of the Pedro Miguel locks.

We would like to thank the rest of the employees at the Panama Canal Authority, Building 706, for their hospitality during the project's eight weeks.

We would like to thank our project advisor, Professor Tahar El-Korchi, Department Head of the Civil and Environmental Engineering Department at Worcester Polytechnic Institute, for his support and guidance throughout this project.

We would like to thank Professor Paul Mathisen of Worcester Polytechnic Institute for all of his advice and knowledge about hydraulics and fluid mechanics.

We would like to thank all of the alumni from WPI who currently reside in Panama for their hospitality during our stay in Panama.

We would like to thank our family and friends for their love and support during this project.

Finally, we would like to thank Worcester Polytechnic Institute for presenting us with this wonderful opportunity.

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1.0 Introduction

The Panama Canal has been a vital waterway in global trade for almost the past 100 years, connecting the Atlantic and Pacific Oceans. This waterway was completed in 1914 by the United States, who operated the Canal until it was turned over to the Republic of Panama in 1999. Prior to the turnover, the Panama Canal Authority (Autoridad del Canal de Panamá or ACP) was established in 1997 by Panama's National Constitution. The ACP was given the power to manage and operate the Canal; the Canal's contributing watershed; and all of the related facilities upon the turnover. Among their numerous facilities, the ACP manages the Paraíso Pumping Station and the Miraflores Potable Water Plant, which currently provides 50 million gallons per day (MGD) of potable water to Panama City.

Currently, the pumps at Paraíso are operating mainly to overcome the friction in the existing pipelines that connect to the Miraflores Potable Water Plant. The pumping energy that is used to overcome this friction is significant, causing the ACP to label the station as inefficient. If the Pumping Station was operating efficiently, the ACP would not have to waste electricity that it is generating to run the pumps. This use of energy reduces potential revenue for the ACP because the energy used to operate these pumps could be sold to the electric company to earn a profit. There is a need to improve the efficiency at the Plant, so it becomes necessary to consider alternatives to alleviate the need for pumping power for the current demand scenario. It is also expected that the demand of potable water from the Miraflores Plant will increase in the near future due to the population growth of the serviced areas.

The purpose of the overall project was to analyze the energy usage of the Gamboa and Paraíso Raw Water Pumping Stations and identify possibilities for improving the energy efficiency of the entire raw water supply system. The raw water is directed from the Pumping Stations to the Miraflores Potable Water Plant, which is currently operating at 50 MGD but is projected to increase by 40% to 70 MGD in the future. Due to the two current lines operating at capacity, the current inefficiencies of the system have a high cost. An analysis was done by the Department of Water, Environment, and Energy of the Panama

Canal Authority that identified the trends in electrical intensity as compared to the flow from the Pumping Stations. The recommendations of the report titled “Análisis Estadístico del Bombeo de Agua Cruda para la Potabilizadora de Miraflores y Recomendaciones para mejorar su Eficiencia Energética,” (“Statistical Analysis of Raw Water Pumping Water Treatment for Miraflores and Recommendations for Improving Energy Efficiency”) included some alternatives such as the installation of a third line from Paraíso to the Miraflores Plant and the connection of a rainwater catchment line located along the Pedro Miguel River to the raw water system.

The purpose of this Major Qualifying Project was to perform a feasibility analysis that would determine the best alternatives to increase pumping efficiencies, reducing power usage and electricity demand, and to increase the capacity of the Paraíso Pumping Station to provide water to the Miraflores Potable Water Plant if a demand increase becomes necessary in the future. One of the main proposed solutions was the installation of either a 30 or 36 inch diameter third fiberglass pipeline connecting the Paraíso Pumping Station and the Miraflores Potable Water Plant, as well as considerations for a connection to the Pedro Miguel rainwater catchment. The set up costs and the operating costs of all the alternatives were compared to determine the most cost effective alternative for a given demand scenario.

2.0 Background

2.1 Canal Background

2.1.1 Panama Canal History

The Panama Canal is a vital waterway to the international trade and shipping industry. The construction of the Canal through the narrow Isthmus of Panama created an alternative route for ships to travel between the Atlantic and Pacific Oceans instead of travelling around Cape Horn on the southern tip of South America. The path of the Canal through the isthmus can be seen in Figure 1.

Figure 1 - The Panama Canal on a Map of Panama (Commonwealth of Australia, 2001)

Initial construction of the Canal was first attempted by the French in 1881. The French planned to build a sea-level canal that would pass straight through the narrow isthmus. Many problems plagued the French efforts including diminishing funds, unexpected problems with excavation, and diseases like Malaria killing many workers. By 1898, the French efforts had come to a halt (Panama Canal Authority, 2012b). The French Panama Canal Company sought ways to abandon the project and decided to approach the United States about purchasing the project and continuing the construction. It took the United States five years before deciding to overtake the Panama Canal project.

After Panamanian Independence from Columbia in 1903, the United States negotiated a treaty with Panama to take over the construction of the Panama Canal. In 1904, the United States paid Panama $10 million and began work on the canal. At a final cost of $375 million, the Canal was finally completed and opened for operation in 1914 (Panama Canal Authority, 2012c; Panama Canal Authority, 2012a). The total length of the Canal is approximately 50 miles from the Atlantic coast to the Pacific coast. It takes a typical vessel between eight and ten hours to make the trip. The canal can accommodate between 30 and 40 ships on a daily basis (Panama Canal Authority, 2012d).

The Canal consists of three locks, broken up into six lock chambers. These locks are used to raise and lower ships from sea level to the level of Lake Gatun, a difference of approximately 85 feet (Panama Canal Authority, 2012d). There is one lock on the Atlantic side which connects the Atlantic Ocean to Gatun Lake, known as the Gatun locks. The Gatun locks have three consecutive chambers. There are two locks on the Pacific side. The larger of the two locks is known as the Miraflores locks, which connect the Pacific Ocean to Miraflores Lakes. The Miraflores Locks consist of two chambers. The other lock is known as the Pedro Miguel locks, which connects the Miraflores Lake to Gatun Lake. The Pedro Miguel locks have one chamber. Currently, there are two lanes of locks, meaning that at the location of each lock chamber, there are two lock chambers side by side allowing for two ships to pass in opposite directions or two ships to travel together in the same direction. The layout of the locks within the Canal can be seen in Figure 2.

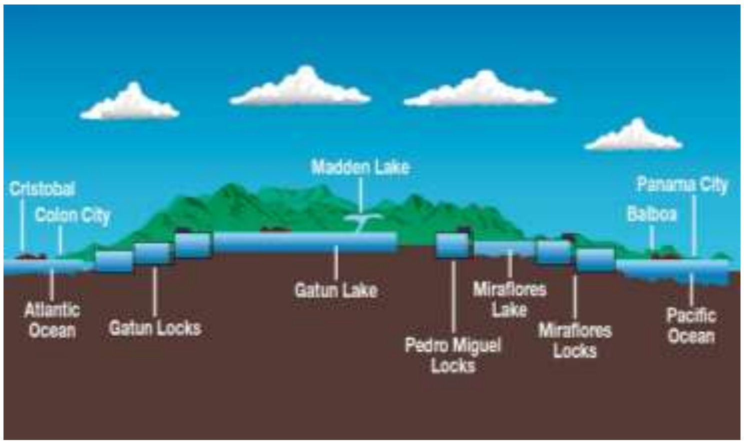


Figure 2 - Layout of the Locks within the Canal (Profile of the Panama Canal System, 2011)

Each lock chamber is 110 feet wide and 1000 feet long. Water to fill the locks is drawn from Gatun Lake. Gatun Lake is approximately 163.38 square miles and was created during the construction of the Canal. The gates to the lock chamber close and seal once the ship(s) is in place and the process of filling or draining the chamber begins. This process takes between eight and twenty minutes depending on the flow entering or leaving the chamber (Panama Canal Authority, 2012d). The process to fill the locks is completed using gravity, which allows the water to flow naturally “downhill” and fill the locks. The valves that control this flow require electricity to operate (Panama Canal Authority, 2012f). The gates of the locks are opened and closed through the use of hydraulic struts. The changeover to hydraulic struts began in 1999 and took several years to complete. Prior to this time, the gates were opened and closed by a large drive wheel (Panama Canal Authority, 2000).

2.1.2 Management of the Panama Canal

As part of the United States takeover of the Canal construction project, the Isthmian Canal Commission was created in 1899 (Global Security, 2011). The original commission was formed to study the feasibility of continuing the canal through Panama and to explore other viable options for the construction of a canal somewhere in Central America. Originally, the commission wanted to build a canal through Nicaragua (Panama Canal Authority, 2012c; Panama Canal Authority, 2012g). Later, upon determining that the French construction area was the most viable option, the second Isthmian Canal Commission was formed. The duty of this new Commission was to maintain a healthy and contented work force during the construction period (Panama Canal Authority, 2012h; The U.S. National Archives and Records Administration, n.d.).

Upon the United States completion of the Canal construction, the Isthmian Canal Commission dissolved and a Canal Zone Government was created to operate as the governing body of civil matters within the Canal Zone. Through the establishment of this government, the United States created positions for a governor, secretary, treasurer and auditor. The government also created a judicial department with several courts, a postal service and an educational system for the Canal Zone (Panama Canal Authority, 2012a;

The U.S. National Archives and Records Administration, n.d.). The Panama Canal Company managed the operations of the Canal. Both the Canal Zone Government and the Panama Canal Company were operated by the United States. In 1914, an administrative building was built to house the operations of the Panama Canal Company. This building is still standing today and is known as the Administrative Building for the Panama Canal Authority.

Since the opening of the Canal, many Panamanians thought that the Canal should be turned over from the United States to the Republic of Panama. During U.S. operations, growing resentment among the Panamanians resulted in several protests. In one instance, several Americans and Panamanians died in January of 1964 after Panamanian students attempted to raise the flag of Panama at Balboa High School, within the Canal Zone (Panama Canal Authority, 2012i). In September of 1977, Leader Omar Torrijos of Panama and President Jimmy Carter of the United States signed the Torrijos-Carter Treaty, which negotiated the transfer of control of the Canal from the United States to Panama. The transfer process was set to start in October of 1979 and take 20 years, allowing for a slow, gradual transition. The treaty also defined the transition from the Panama Canal Company and Canal Zone Government to the Panama Canal Commission. The Panama Canal Commission was formed to act as the facilitating body during the transfer of the Canal. The Panama Canal Commission consisted of a total of nine members, made up of Panamanians and United States citizens. Initially, there were five members from the United States and four Panamanians on the Commission. The administrator of the Commission was an American and the assistant administrator was a Panamanian. These two roles were maintained for the first ten years of the transition process. The second ten years prompted a role reversal, where a Panamanian was the administrator and an American was the assistant administrator (Panama Canal Authority, 2012j).

In preparation for the official handover on December 31st, 1999, the Panama Canal Authority (Autoridad del Canal de Panamá or ACP) was created in 1997. The ACP was created after an amendment to the National Constitution of Panama established it as an autonomous part of the Panamanian government (Panama Canal Authority, 2012k). The ACP manages its own finances and has its own set of rules and laws, which may supersede Panamanian law for matters pertaining to the Canal. The role of the ACP is to manage Canal operations, to perform maintenance on the Canal and the locks when necessary, and to make improvements to the Canal and its operations. Currently, such improvements include expanding the locks and upgrading Lake Gatun operations to better accommodate the needs of the current maritime market.

2.1.3. The Panama Canal Authority

The Panama Canal Authority (ACP) is controlled by an administrator and a deputy administrator. These two positions are under the supervision of a board of directors, which features eleven members. Nine of the directors are appointed by the President of the Republic of Panama with the consent of the Cabinet Council and require ratification by an absolute majority of the members of the Legislative Assembly. One director is designated by the Legislative Branch. The final director is appointed by the President, not requiring consent of the Cabinet Council or ratification by the Legislative Assembly, and is designated as the Chair of the Board. This director will have the rank of Minister of State for Canal Affairs (Panama Canal Authority, 2012k).

The Panama Canal Authority has several goals. One of their goals is to be a world leader in services to the maritime industry. This goal guides the ACP to operate the Panama Canal at its fullest potential and to keep it up to date with technological advances. Another goal is to be a world leader in sustainable development for the conservation of the Panama Canal Watershed. The ACP is working to achieve this goal through the current expansion project, which works on conserving water loss in the locks. The ACP also strives to be a cornerstone of the global transportation system by providing exceptional service through the Panama Canal. The final goal of the ACP is to be a driving force for the progress, development and growth of Panama. This goal is meant to establish a base for the ACP to help develop the economy and therefore the growth of Panama by utilizing the Canal (Panama Canal Authority, 2012l). It is with these goals in mind that the ACP conducts its work.

The Panama Canal Authority began immediate work to make changes and upgrade the Panama Canal after the handover. When the ACP first took over control of the Panama Canal on December 31st, 1999, they increased the tolls that ships were required to pay to pass through the Canal. Later, in 2002, the ACP restructured the way the tolls were calculated. Originally, tolls were strictly based on tonnage. The new structure included different rates for different types of ships, size of ships, and type of cargo that is onboard (Panama Canal Authority, August 2012). The aim of these changes was to appease customers and to attract more ships to make the journey through the Canal, thus increasing the income to the Country of Panama.

In accordance with Article 18 of the Panama Legislative Assembly Law No. 19 ("Whereby the Panama Canal Authority is Organized"), different departments were formed to suit the different needs that the Canal presented to both the Authority and to the country of Panama. Currently there is an Environmental Department, an Administrative and Finance Department, an Engineering and Program Management Department, an Operations Department, a Human Resources Department and a Planning and Business Development Department (Panama Canal Authority, October 2012). These departments all work on different aspects that help the ACP and the Canal operate to its fullest potential. Each of the departments works on projects to improve Canal operations. A few examples include efficiency of raw water treatment and delivery, efficiency of energy, and alternative energy sources, which are all being completed by the Environmental Department. Some larger projects are combined efforts between multiple departments such as the Panama Canal Expansion project.

2.1.4 Panama Canal Expansion Project

Once the Panama Canal Authority (ACP) took power in 1999, they decided to pursue options to expand the Panama Canal. Some of the reasons to pursue expansion included significant traffic congestion passing through the Canal due to an increasing number of ships that attempt to pass through every year and the demand to accommodate larger ships. A plan for expansion was approved in 2006 by the population of Panama. There were four goals set in place for the expansion project. According to the Panama Canal Authority, these goals were:

1) Achieve long-term sustainability and growth for the Canal's contributions to Panamanian society through the payments it makes to the National Treasury;
2) Maintain the Canal's competitiveness as well as the value added by Panama's maritime route to the national economy;
3) Increase the Canal's capacity to capture the growing tonnage demand with the appropriate levels of service for each market segment;
4) Make the Canal more productive, safe, and efficient. (Panama Canal Authority, 2006)

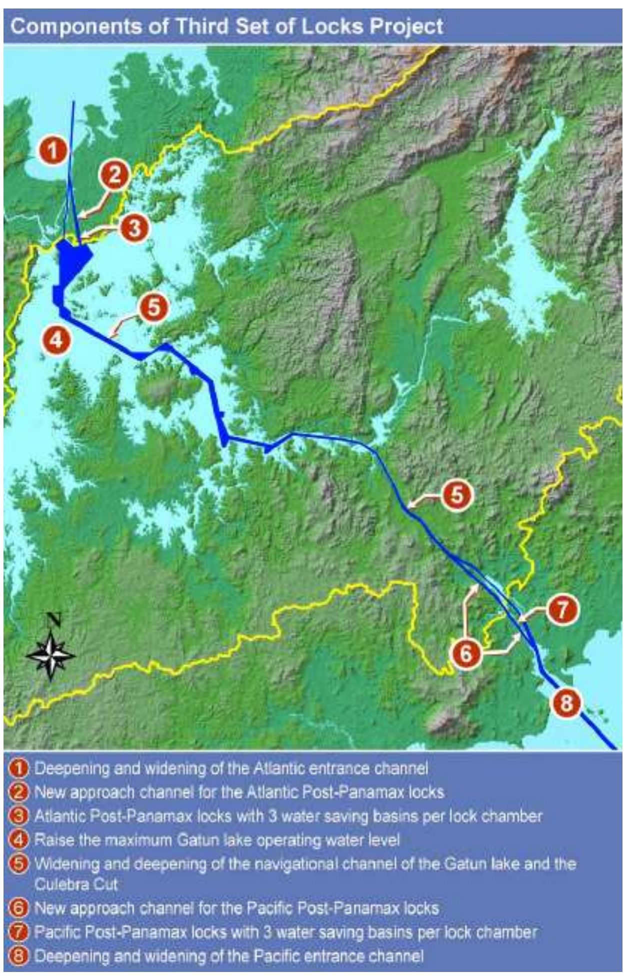


Figure 3 - Components of the Third Set of Locks Project (Panama Canal Authority, 2006)

The expansion will add a third "lane" of locks to the Canal system, which will be able to accommodate larger ships. The components of the expansion project can be seen in Figure 3.

The total cost of the project, including labor, excavation and parts for the new locks will be approximately $5.25 billion. While this is a large initial capital investment, the projected revenues from the Panama Canal after the completion of the expansion project are expected to be $6,000 million after the first 11 years of operations (Panama Canal Authority, 2006).

One of the largest components of the expansion project is the construction of a third lane of locks. The third lane will add a new lock system to each end of the Canal. On the Atlantic Ocean side of the Canal, the new set of locks will run parallel to the Gatun locks. This set of locks will contain 3 chambers, used to raise ships from the Atlantic Ocean to the level of Gatun Lake. On the Pacific Ocean side of the Canal, the new set of locks will bypass the Miraflores and the Pedro Miguel locks. This set of locks will also include 3 chambers, which will raise and lower ships between the Pacific Ocean and Gatun Lake. Each chamber of the new locks will be 1,400 feet long by 180 feet wide and 60 feet deep. This larger size will allow Post-Panamax ships to travel through the Canal (Panama Canal Authority, 2006). Post-Panamax is a term used to classify ships. With the current lock size, the Panama Canal can only accommodate Panamax ships. Post-Panamax ships are larger than Panamax ships, necessitating the new larger locks.

The new lane of locks will be designed differently than the original locks. The original locks feature gates that open and close on hinges. The new locks will feature rolling gates which have become a standard for many locks of this size around the globe. The layout of the new rolling gates can be seen in Figure 4.

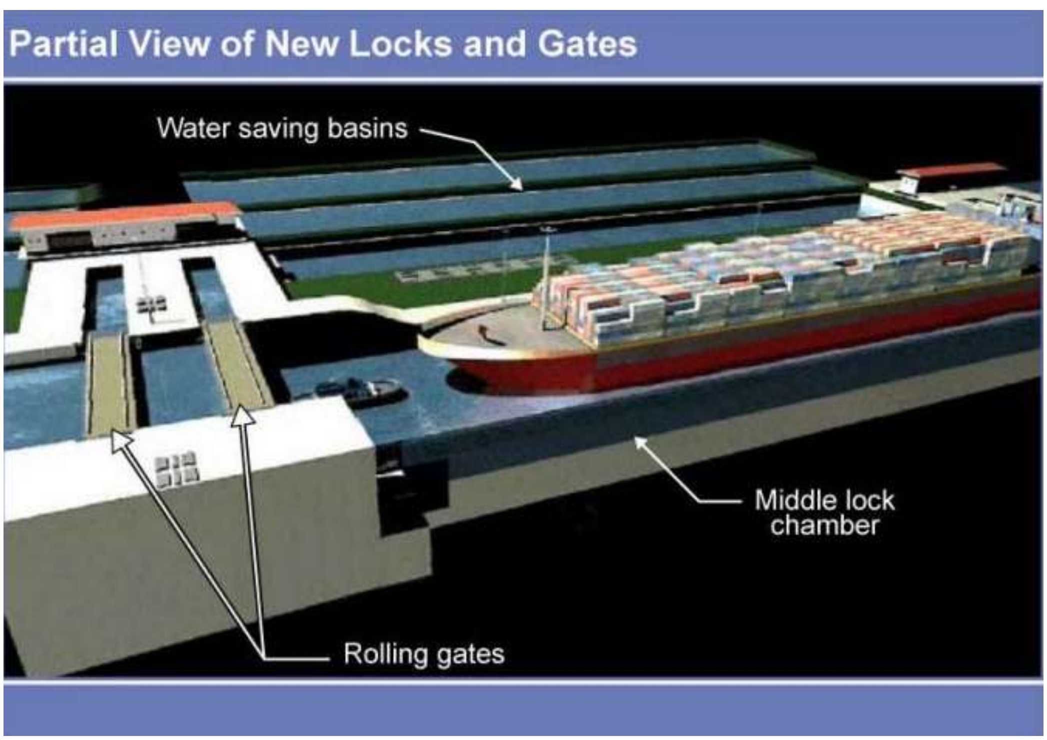


Figure 4 - Layout of New Lock Chambers with Rolling Gates (Panama Canal Authority, 2006)

The advantage to rolling gates is that their storage area acts as a dry dock that allows for maintenance instead of having to physically remove the gates to perform any repairs. Another advantage of the rolling gates is that they run perpendicular to the lock chamber. This design allows ships to be positioned closer to the gates than the current lock design. With the hinged gates in the current locks, ships must be positioned to allow room for the gates to swing open and closed.

The current locks use locomotive engines to load the ships into the sections. These locomotives, known as mules, do not push or pull the ships. They merely act as guidance to keep larger ships properly positioned. Figure 5 is an image of a mule at the Pedro Miguel locks.

Figure 5 - Locomotive Guiding a Ship in the Pedro Miguel Locks (Photo Taken by Shelby Miller, 2012)

Due to the increased size and tonnage of Post-Panamax ships, a large number of locomotives would need to be utilized to position the vessels within the chambers. Instead, the new locks will employ the use of tugboats to aid in the positioning of the ships. A tugboat can be seen in Figure 4, guiding a container ship into place within a lock chamber (Panama Canal Authority, 2006).

As one of the goals of the Authority is to protect the Panama Canal watershed, the use of more water in the new larger lock chambers is a concern to the ACP. The larger size of the new locks will mandate that a larger volume of water is used to fill each chamber. In order to help reduce the water that lost during Canal lockages, the new lock chambers will feature water saving basins. These basins will help recycle some of the water that is used in the lockage process. Each lock chamber will feature three basins (Panama Canal Authority, 2006). The proposed set up for the new locks with water saving basins can be seen in Figure 6.

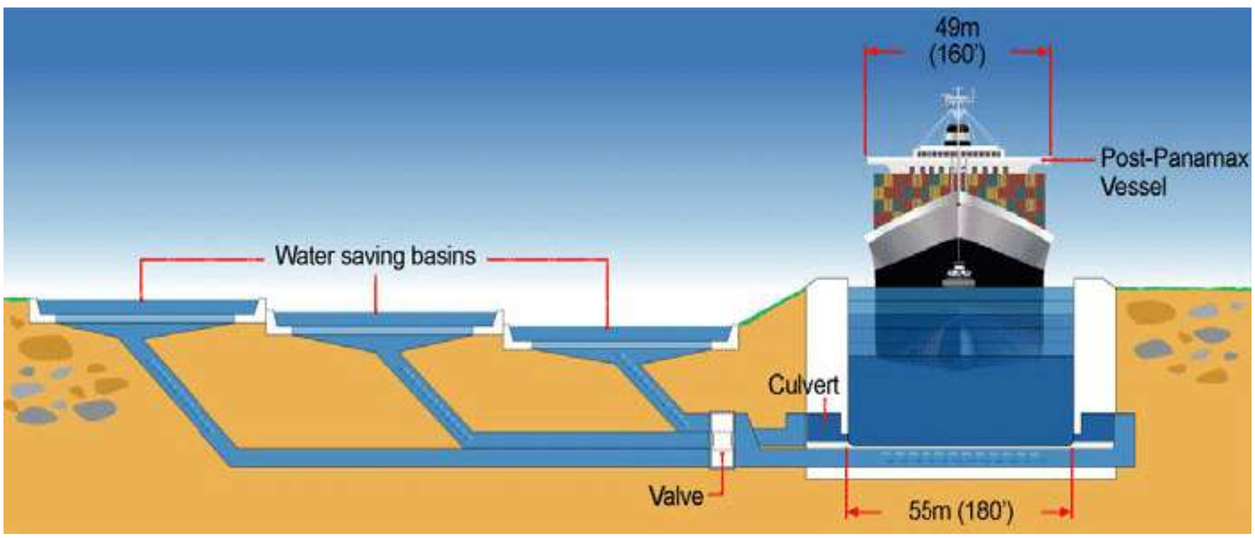


Figure 6 - Cross Section View of New Locks with Water Saving Basins (Panama Canal Authority, 2006)

Three basins were chosen over other possible configurations due to the water yield, efficiency of saving water and construction cost. Alternative numbers of basins would either have a smaller water saving efficiency or a higher construction cost. These basins will be gravity fed and will recycle about 60% of the water that is used to fill one chamber. Recycling 60% of the water means that even though each chamber requires a larger volume to operate, the new lock chambers will use 7% less freshwater than the current locks chambers (Panama Canal Authority, 2006).

This ability to conserve water that would have been removed from the Panama Canal watershed is important to the conservation of the watershed. This conservation is being monitored by the Environmental Department of the ACP and by the Interagency Watershed Commission for the Panama Canal (Comisión Interinstitucional de la Cuenca Hidrográfica del Canal de Panamá or CICH) (Panama Canal Authority, 2012m).

2.1.5 The Panama Canal Watershed

The Panama Canal Watershed has a surface area of approximately 5,528 square kilometers. The boundary of the watershed was officially defined in 1999 by Law 44. Law 44 is a Panamanian Law that was developed to establish basic parameters governing the management of the Canal watershed (Canal Authority, 2012m; Winner, 2005). The boundary of the watershed is outlined in red on Figure 7.

Figure 7 - Boundary of the Panama Canal Watershed (Winner, 2005)

The watershed is made up of three different regions. The southern region is the Miraflores Lake sub-basin, which is the smallest portion of the watershed. The Alajuela Lake sub-basin is the eastern region of the Canal. The last region of the watershed is Lake Gatun in the central and western region, the largest part of the watershed. The watershed provides the water that flows through the Canal as well as the water that is used in the lockages of each vessel that traverses the Canal. Each vessel that travels through the Panama Canal requires approximately 52 million gallons to complete passage through all 6 lock chambers (Panama Canal Authority, 2012m).

The watershed is also the major water source for the Republic of Panama. With a growing demand for potable water in the areas surrounding the Canal, insufficient management of this vital resource could result in a lack of sufficient and potable water for the citizens of Panama. In order to accommodate this growing need, the Panama Canal Authority is looking to expand the capacity of one of their water supply systems, the Paraíso Pumping Station and Miraflores Potable Water Plant.

Figure 8 - Two 30 Inch Pipelines Leaving Paraíso Pumping Station (Photo taken by Cindy Lin, 2012)

2.1.6 Paraíso Pumping Station and the Miraflores Potable Water Plant

The Paraíso Pumping Station pumps raw water up from the Panama Canal in the Gaillard Cut, a few hundred meters north of the Pedro Miguel locks on the Pacific side of the Canal. Raw water is natural water found in the environment. The term "raw" indicates that it has not yet been treated and is not suitable for human consumption (Jones, 2008). The raw water at Paraíso is then sent to the Miraflores Potable Water Plant where it is treated to a level that is acceptable for human consumption. The current system utilizes two 30 inch pipelines between the Paraíso Pump Station and the Miraflores Potable Water Plant, known as the North line and the South line. The lines were named as such due to their relation to one another. The North pipeline can be seen on the right leaving the Paraíso Pumping Station in Figure 8.

The Station and the Plant are located approximately four and a half kilometers from one another. Figure 9 is a drawing which shows the Paraíso Pumping Station and the Miraflores Potable Water Plant in yellow, the North line in orange and the South line in green. To the left, there is a pink line that carries water from the Gamboa Pumping Station (also shown in yellow on the figure). The line from Gamboa meets up with the North line.

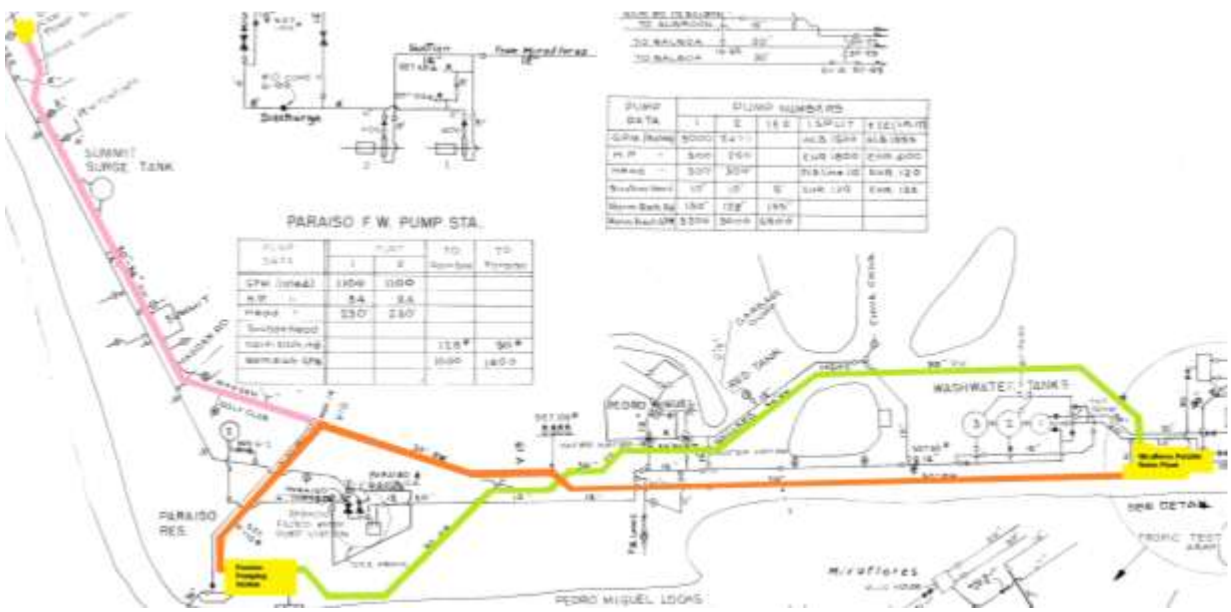


Figure 9 - Figure showing Paraíso Pump Station and Miraflores Filter Plant (Panama Canal Authority, 1981)

This water supply system was first constructed in 1913 and went into operation in March of 1915 (Panama Canal Company, 1955). Originally, there was only the North line ran between Paraíso and Miraflores. The South line was added later in 1964 as an upgrade to the Plant, which also increased the capacity of the system from 25,000 gpm to 35,000 gpm (Water Supply Increase Set, 1964). There are five vertical pumps at the Paraíso Pumping Station, which can be seen in Figure 10.

Figure 10 - The Five Vertical Pumps at the Paraíso Pumping Station (Photo taken by Cindy Lin, 2012)

Typically, not all five pumps are running. They extract water from Lake Gatun and pump water from Paraíso to Miraflores. Once the water reaches the Miraflores Potable Water Plant, it goes through several steps of treatment. The first step is the water aeration process. Aeration is used to add oxygen to the water and to remove dissolved gases (CO_2, H_2S) and volatile organic compounds (Bortman, Brimblecomb & Cunningham, 2003). The aeration process at the Miraflores Plant can be seen in Figure 11.

Figure 11 – Water Aeration Process at the Miraflores Filter Plant (Photo taken by Shelby Miller, 2012)

The second step is the addition of different chemicals. The Miraflores Plant utilizes several different chemicals in its treatment process. Chlorine is added to the water as a disinfectant that kills bacteria and algae. Fluoride is added to help prevent tooth decay. Figure 12 shows an area where chemicals are added to the water.

Figure 12 - Chemical Additives at the Miraflores Potable Water Plant (Photo taken by Cindy Lin, 2012)

After the addition of these chemicals, the water is mixed. The next step is the flocculation process. In the flocculation step, water flows into tanks that have large, slow moving paddles. These paddles slowly mix the water and bring particles together (Bratby, 2006). Aluminum sulfate polymers are added to the water during the flocculation phase to assist in the process. As the particles join together, they make larger particles which are known as flocs. Figure 13 shows the flocculation area. Small flocs can be seen in the water.

Figure 13 - Suspended flocs at the Miraflores Potable Water Plant (Photo taken by Cindy Lin, 2012)

As flocs increase in size, they increase in weight and sink to the bottom of the tanks where are removed as sludge (Bratby, 2006). The next step is the sedimentation process where suspended solids collect at the bottom of sedimentation tanks and are separated from the water. The sedimentation process is also used to reduce the turbidity in the water. After the sedimentation process, the water flows through a series of filters. At the Miraflores Plant, the water first flows through anthracite coal, and then sand and then gravel. A sedimentation basin can be seen in Figure 14. After filtering, the water is considered potable. Chlorine is sometimes added again to help disinfect the water and kill any bacteria that the water may contact as it flows through the distribution pipelines.

Figure 14 - Sedimentation Basin at the Miraflores Filter Plant (Photo taken by Shelby Miller, 2012)

The drinking water that is produced by the Miraflores Potable Water Plant is purchased by the National Institute of Aqueducts and Sewers (El Instituto de Acueductos y Alcantarilla dos Nacionales or IDAAN), which responsible for supplying drinking water to the entire Republic of Panama.

The current Pumping Station, with the two 30 inch lines, yields approximately 50 millions of gallons per day (MGD) of potable water. These pipelines are at full capacity and require pumping energy to help overcome the friction loss that occurs within the pipelines. The pumps are considered to be inefficient, working against a significant amount of friction to produce the necessary flows. The ACP would like to find a different solution that will reduce the energy use, a source of revenue in the future.

An analysis of a third water line was conducted to determine if the addition of a third line to this system will reduce the pumping energy expended on the two current lines. The amount of energy reduction was also determined. An analysis determined if the Pumping Station will be able to meet projected future demands for 70 MGD with the addition of a third line. Other considerations included whether or not a rainwater catchment system from the Pedro Miguel sub-basin would be a viable option to increase the flow in the system through a gravity fed line that carries water down the hill. Prior to making a final decision, the ACP must assess a number of technical and economic consideration including capacity of the pipes to handle the flow, corresponding pipe diameter, whether or not a pump will be needed to accommodate the varying flow that would result from a rainwater catchment system, and ultimate cost.

2.2 Technical Background

The existing flow to the Miraflores Potable Water Plant is made possible by a series of pumps and pipelines that feed raw water from the Paraíso Pumping Station and the Gamboa Pumping Station. The Pumping Stations, pipelines, and treatment facility are all a part of a water distribution system, managed and operated by the Panama Canal Authority (ACP). In order to design and analyze each of these components, an understanding of a water distribution system, how energy is measured in water systems using Bernoulli’s equation, pump design, and demand curves will be necessary.

2.2.1 Water Distribution Systems

For major urbanized areas, a water distribution system is needed to provide potable water as well as wastewater removal for consumers. Due to the population densities of these urban areas, it is not feasible to utilize a well and septic tank system. A water distribution system often consists of a complex network of pumps, pipelines, storage tanks, and treatment facilities. It requires a source of water as well as a discharge point. Operating and maintaining these systems has been the responsibility of public utilities and government agencies in each area, often with some base quality regulations from the government.

In water distribution systems, there are components necessary for functional operation including built infrastructure and natural features. Water begins its journey in a drainage basin, a section of land that receives water from runoff, rainfall or otherwise that converges in rivers, streams, lakes, and groundwater. At this point, the water is considered raw water. The water utility will then take the raw water from the source, often a large surface water body like a reservoir or a lake, into the system. The water is directed to a water treatment plant, where the quality of the water is improved by removing toxic or undesirable chemicals, bacteria, and microorganisms. Following the treatment, the water is considered potable and can be used as drinking water (Loucks et al., 2005). Then, the water is distributed to the customers within the network based on varying demands.

Once the water is used in the system, it is considered wastewater and is collected through a different system to be treated at a wastewater treatment plant in place of a traditional septic system. Often, wastewater treatment plants will also take urban runoff and infiltrating groundwater collected through storm drains and sewers and treat it as if it is wastewater, though often does not require as much treatment as wastewater. The goal of the plant is to reduce pollutants to an acceptable level before the water is discharged into the natural environment (Loucks et al., 2005).

Satisfying potable water demands often prove to be a challenge for water utilities. It often requires balancing the right levels of water in the system with a changing rate of demand from the consumers. Demand changes throughout the day and seasonally while potable water treatment plants produce at a fairly constant rate. Utilities have to be careful not to exhaust the source when trying to fulfill the demand as that can degrade the water quality. Often, the entire system upstream of water usage is involved in finding that balance through the use of Pumping Stations, storage tanks, pipelines, and regulating valves (Loucks et al., 2005).

2.2.2 Gravity and Pump Flow

Flow is an important part of water distribution systems, whose various components are linked by pipelines and aqueducts. In order to understand the flow from one source to another, it is important to know the energy of a system at any given location compared to another, often determined through Bernoulli's equation. This energy is often expressed as a height, known as the head. The main source of head loss in much of the system is due to the friction between the water and the pipe. Head in the system can be gained by a pump or a series of pumps. This is mathematically represented by the following equation:

$$\frac{p_1}{\gamma} + z_1 + \frac{{v_1}^2}{2g} + H_G = \frac{p_2}{\gamma} + z_2 + \frac{{v_2}^2}{2g} + H_L \tag{2.1}$$

In equation 2.1, the subscripts 1 and 2 indicate the two sites, p is the pressure, z is the elevation, v is the velocity, γ is the specific weight of the fluid, g is the gravitational constant, and H_G and H_L represent head gain and loss, respectively (Loucks et al., 2005).

When determining the head loss due to friction to understand if the flow is open channel flow or pressurized pipe flow. For open channel flow, Manning's equation can be used to determine head loss. For pressurized pipe flow, the Hazen-Williams Equation or the Darcy-Weisbach Equation can be used to determine head loss. When using the Darcy-Weisbach equation, the Reynolds number is important to identify whether flow is laminar or turbulent. If the energy of the system at the discharge point or at a connection point is higher than its starting point, the water will not flow to the second point. In that situation, a mechanism is needed that will raise the head, such as a pump.

The energy of a system can be graphically represented through the energy and hydraulic grade line as can be seen in Figure 15. First, an elevation datum needs to be determined, a line that represents elevation zero. Then, the elevations from the datum to the points within the system are the first values depicted. For open channel flow, the elevation is considered to be at the top of the water. For pressurized pipe flow, the elevation is considered to be at the center of the pipe system. Then the value of pressure over specific weight is added to the elevation values. For all points along the system, the sum of those two values is considered the Hydraulic Grade Line (HGL). Finally, added to that is the velocity term, expressed as $v^2/2g$, to the HGL. This is the Energy Grade Line (EGL). For both lines, the loss due to friction must also be considered. Note that Figure 15 is a simple example a diagram of energy in pressurized pipe flow (Loucks et al., 2005).

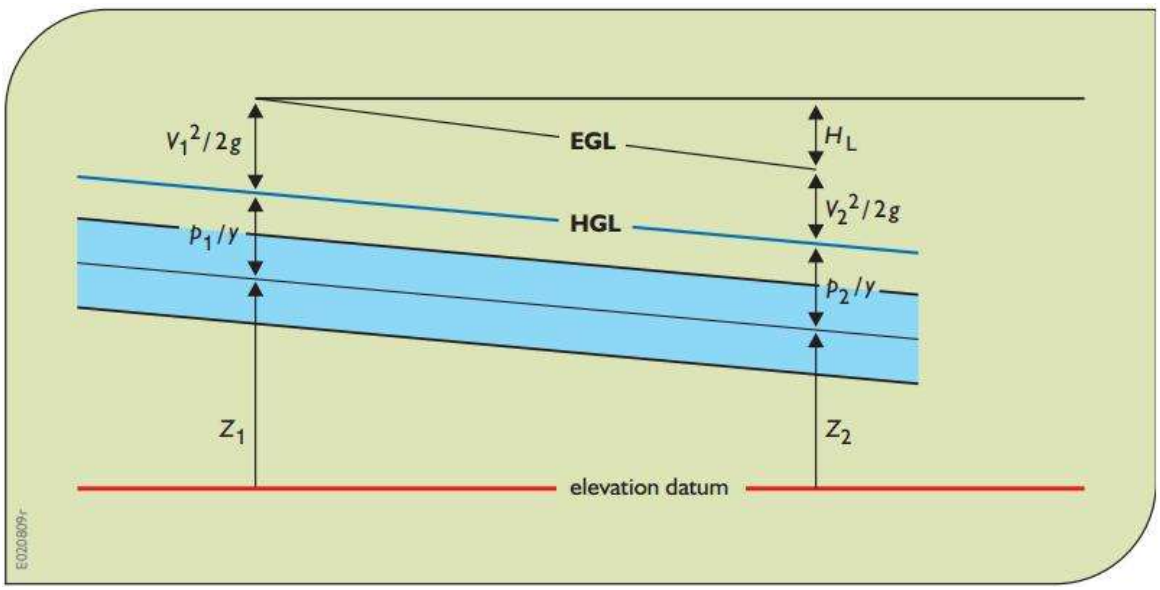


Figure 15 - An example depicting the Energy Grade Line (EGL) and Hydraulic Grade Line (HGL) for a section of pressurized pipe flow (Loucks et al., 2005)

Depicting more than just pipe flow, all the various losses caused by turns, connections, entrance, valves, changes in pipe diameter, etc., as well as energy added due to pumps, must all be considered when calculating losses for the diagram and the equation. An example depicting the EGL and HGL for a more complex system can be seen in Figure 16.

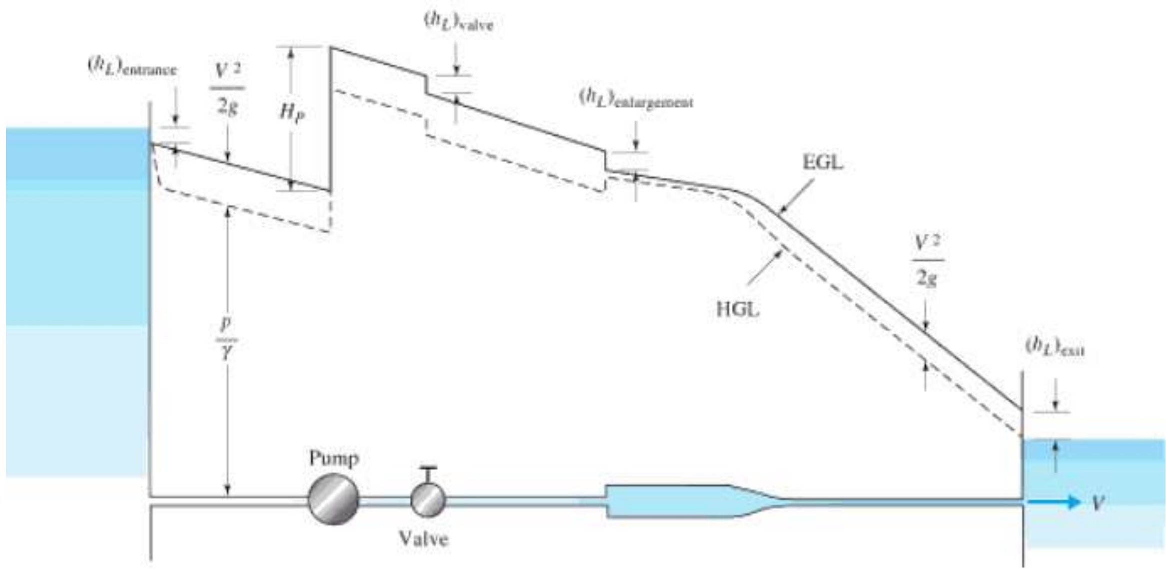


Figure 16 - An example of a system and its corresponding Energy Grade Line (EGL) and Hydraulic Grade Line (HGL) (Potter, Wiggert & Ramadan, 2011)

2.2.3 Pump Usage

It is almost certain that pumps will be needed in a water distribution system. There are many different kinds of pumps but they all provide the same function, to add energy to the system. This energy can be used to add to the height, the pressure, or the velocity of the water, providing flow to desired locations. To calculate the pumping capacity needed in a system, a modified version of Bernoulli's equation is used to determine the head the pump needs to provide (Menon, 2004).

$$H_1 + H_p = H_2 + \sum H_{L_{1-2}} \quad (2.2)$$

In equation 2.2, H_P is the head the pump needs to provide. Note that H_1 and H_2 are defined by the pressure, elevation, and velocity at points 1 and 2.

Once the head needed by a pump is determined, specific pumps need to be considered to determine which is the most suitable for the intended purpose. Pump manufacturers will provide a pump performance or characteristic curve that will depict how much head the system will provide when producing a specific flow value. These will also be paired with a variety of other curves including an efficiency curve, a net positive suction head, and a brake horsepower curve, as shown in Figure 17.

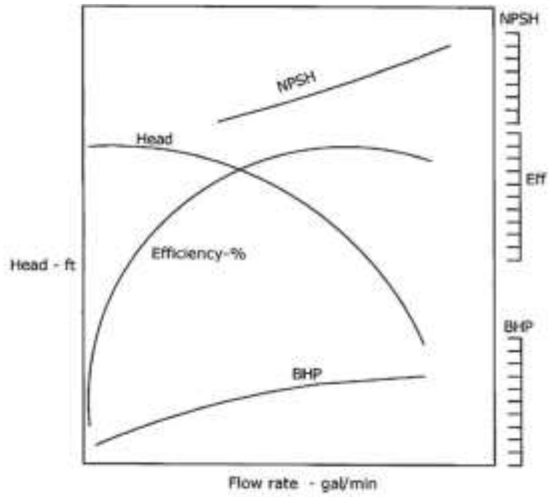


Figure 17 - A basic plot of pump performance curve, efficiency curve, brake horsepower curve, and net positive suction head curve (Menon, 2004)

The pump performance curves that manufacturers provide will also include additional information about varying efficiencies, the performance curve given a different pipe diameter, and the speed of the pump in revolutions for a given time period. Much, if not all, of this information is considered when choosing a pump due to the needs of the system as well as the constraints of the problem, including available land and cost of obtaining, installing, and operating pumps and pipes (Mays, 2000). An example of a plot is shown in Figure 18.

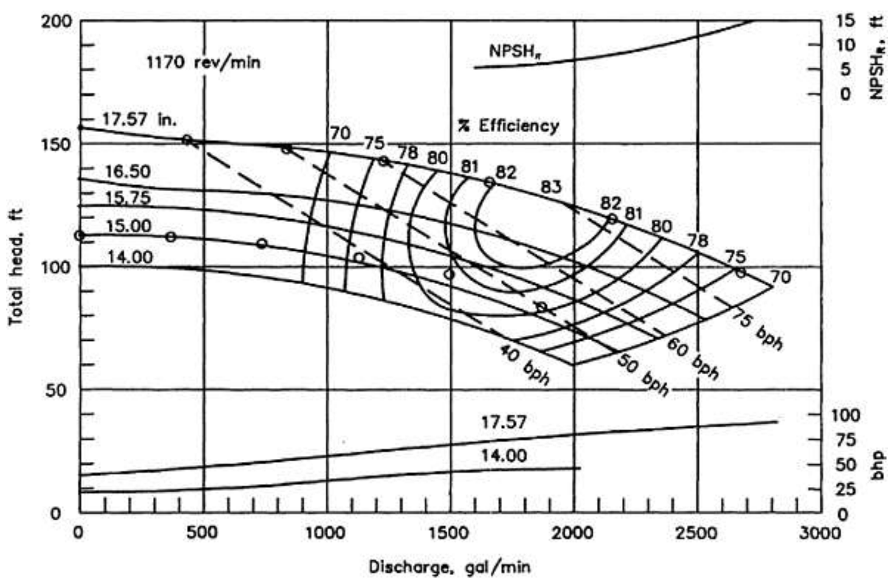


Figure 18 – A Typical Pump Performance Curve (Menon, 2004)

The pump performance curves will be plotted against the system curves. The system curve is defined by Equation 2.3.

$$H_P = dh + kQ^2 \tag{2.3}$$

The head that the pump needs to overcome, H_P, is a function of the flow. Static head, *dh*, is the constant head, equivalent to the difference between H_1 and H_2. The variable k is a constant that represents the total system characteristics that includes minor and major losses. The kQ^2 value can also be computed by the total head loss of the system plus the difference in velocity between 1 and 2. An example of a system curve plotted against the performance curve is shown in Figure 19. Designers will determine which pump suits the needs of the system and the owner, weighing flow, efficiency, and cost (Menon, 2004).

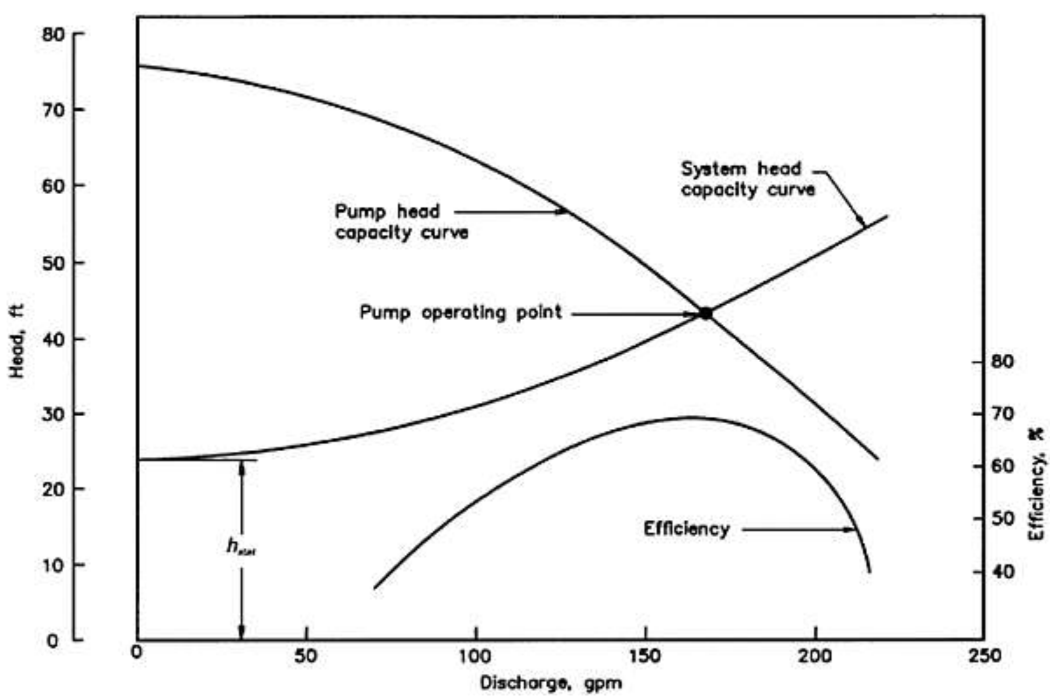


Figure 19 - System and Pump Performance Curves (Menon, 2004)

Singular pumps are often not common in water distribution systems due to the sheer volume of water that these systems must handle. Water distribution systems combine pumps in pump stations and throughout their various facilities including treatment plants and connection points to ensure the necessary flows to operate the system. There are many possible configurations for pump layout, but they break down to two basic forms: series and parallel, shown in Figure 20. A series configuration is used to add head to the system while a parallel configuration is used to add flow to the system. When designing a pump station, the demands of the system need to be determined in order to identify which configurations would be best. Additional pumps produce an updated pump performance curve, as seen in Figure 21 (Menon, 2004).

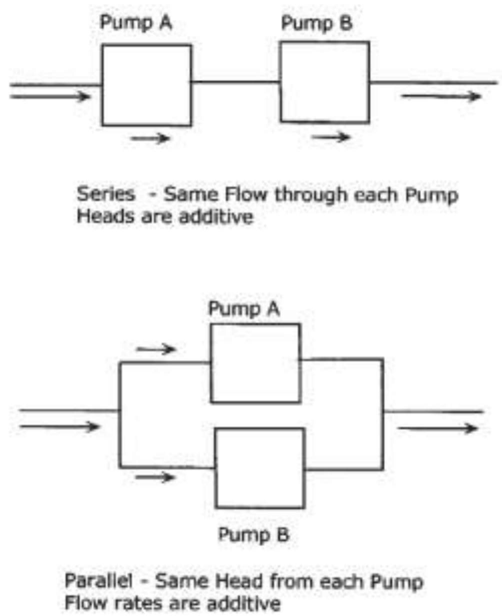


Figure 20 - Two configurations for pumps, one series and one parallel (Menon, 2004)

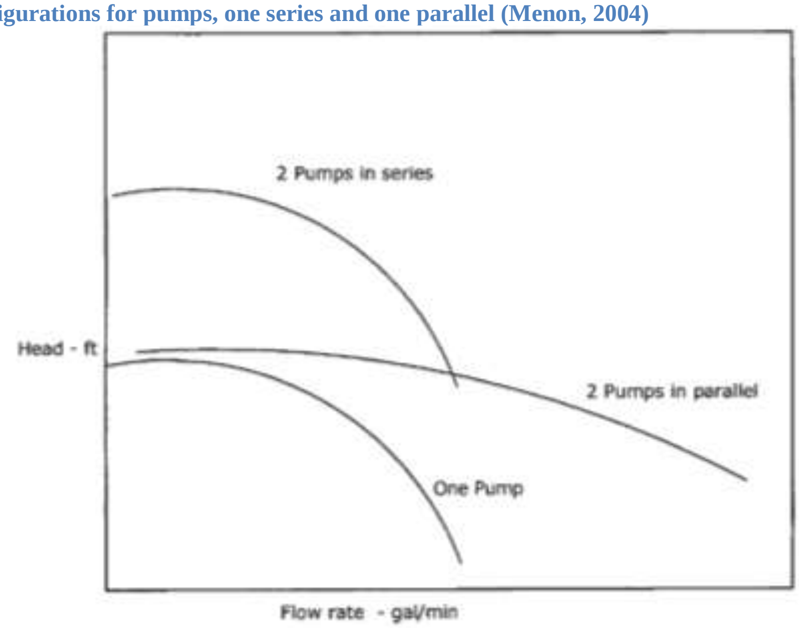


Figure 21 - Pump Performance curve for one pump, two pumps in series, and two pumps in Parallel (Menon, 2004)

The updated aggregate pump performance curve is compared with the system curve in the same manner for the single pump to determine which configuration is most desirable for the conditions (Menon, 2004).

3.0 Methodology

The goal of this project was to perform a feasibility study for the Panama Canal Authority (ACP) on improving the efficiency of the current Paraíso Pumping Station as well as addressing the expected future increased demand for the Miraflores Potable Water Plant. The proposed solution was to install a third raw water line from the Paraíso Pumping Station to the Miraflores Potable Water Plant with an option to connect the third line with a pipeline carrying flow from a rainwater catchment from the Pedro Miguel sub-basin. The feasibility analysis included modeling the current system to define current inefficiencies, designing a third line, designing a connection for the rainwater catchment to the system, projecting future limitations and costs if demand for the Plant increased from 50 MGD to 70 MGD, calculating procurement, installation, and operating costs of each alternative, and comparing all alternatives to determine which option would be most feasible given the ACP's site and budget constraints.

3.1 Identifying Relevant Information

The analysis that was performed required collecting relevant information about the components of the system from ACP files and personnel. Data about flows and volumes of water was gathered from the ACP SCADA system, their automated monitoring system. The SCADA data was also used to compare the results obtained from the model to the actual system. In order to understand the potential energy savings from the model to the system, the two were compared to understand how future projections would mimic the actual system.

The Paraíso Pumping Station has five vertical pumps, which are organized in parallel, to draw water from the source and to overcome friction within the pipelines. Additional information about these pumps was gathered from ACP documents and reports prepared by the ACP and by outside contractors, including the pump and efficiency curves and the actual tested efficiency of the pumps. The duration that each pump is in operation and how they are operated (i.e. on a schedule or by a manual operator) was determined.

Other necessary information that was gathered included information about the pipelines, the two stations (Paraíso Pumping Station and Miraflores Potable Water Plant), and the Pedro Miguel rainwater catchment pipeline. It is known that there are two lines that deliver raw water from the Pumping Station to the Plant. The unknown variables including pipeline diameter, length of the pipe, and material of the pipe were also determined. The diameter was used for flow calculations. The type of material was needed to provide the coefficients for determining the amount of friction within the pipelines. The length of the pipeline was used to determine capacity and friction in the lines. Finally, the elevations above sea level for each of the relevant points of interest were determined from past reports and drawings.

Information about the current demand on the system was also determined. The demand on the system is considered the water needed by the distributor and its customers, defined as outflow from the Potable Water Plant. This demand is completely dependent on the capacity of the pipelines servicing the Miraflores Plant. To perform the necessary analyses, the capacity of the pipelines, or the volume that can flow through the pipeline when full for a given amount of time, was calculated using existing data and models.

3.2 Creating a Model

In order to process all of the information that was gathered, a model was created in Microsoft Excel. This model utilized Bernoulli's Equation and the Hazen-Williams Equation for head loss.

All of the alternatives were analyzed using the same basic method. First, the flows from Gamboa were calculated. Then, the flows from the Pedro Miguel rainwater catchments were calculated for those alternatives that include a connection to the rainwater catchments. For all alternatives, it was assumed that any flow from Gamboa was the result of only one pump in operation, yielding an approximate 5,380 gallons per minute (gpm) flow to the North pipeline. This is the most efficient configuration that includes the Gamboa Pumping Station because it produces the least amount of flow into the North line, requiring less energy at the Paraíso Pumping Station to overcome any added pressure in the North line. The flow from the Pedro Miguel rainwater catchment varied on a monthly basis due to the variation of rainfall in

Panama. Thus, the following analysis was performed twelve times for alternatives that included the rainwater catchment in order to more accurately identify the operating flows and cost for those options.

The outside flow, from Gamboa and the Pedro Miguel rainwater catchment, was subtracted from the water demand for that option to determine the flow needed from Paraíso in order to fulfill that demand. The remaining flow needed is then calculated amongst the three pipelines, the existing two and the proposed third, so that the pump head is equal for the South pipeline and the proposed third line. The pump head for the North line will be lower than the other two due to the inflow from the Gamboa line which is not represented in the model.

The flows within the North and South lines for any given alternatives analysis were calculated using the same ratio of flow between the lines, calculated during the analysis of the current configuration. The lengths of the North and South lines were determined using the ACP Geographic Information System (GIS) software. The length of the third line was determined by taking the shortest length that followed a combination of the existing North and South pipelines. In order to determine the proper pipeline flows for each alternative other than the current configuration, the following equations were used.

$$Q_N + Q_S + Q_3 + Q_G + Q_{RW} = Demand$$

3. 1

$$\frac{Q_N}{Q_s} = 0.984$$

3. 2

$$H_{p_S} = H_{p_3}$$

3. 3

Equation 3. 1 describes the flows that total the demand, where Q_N is flow in the North line, Q_S is flow in the South line, Q_3 is flow in the third line, Q_G is flow in from Gamboa at a constant 5280 gpm, and Q_{RW} is flow from the rainwater catchment, if applicable for that alternative. Equation 3. 2 describes the ratio between the flow in the North and South lines. Equation 3. 3 equates the pump head in the South line and the third line. Because the elevation difference is the same for both lines, the head loss due to friction would be the same. Then, the three unknown flows were calculated from these three equations.

Then, the friction head loss is calculated for each pipe using the modified Hazen-Williams Equation from the flow in that given pipe based on the physical characteristics of the pipe, such as the length and the material of the pipe. The unmodified Hazen-Williams Equation for calculating velocity is shown below as equation 3. 4. The modified Hazen-Williams Equation for calculating head loss is shown below as equation 3. 5.

$$v = kCR^{0.63}S^{0.54}$$

3. 4

$$h_f = \frac{10.67LQ^{1.85}}{C^{1.85}d^{4.87}}$$

3. 5

Where k is the conversion factor for the unit system (k = 0.849 for SI units), C is the roughness coefficient, R is the hydraulic radius and S is the slope of the energy line. In equation 3. 5, h_f is the head loss over a length of pipe, L is the length of the pipe, Q is the volumetric flow rate and d is the inside pipe diameter. Equation 3. 5 was used to find the head loss in each pipeline.

For the analyses of the rainwater catchment alternatives, an additional step was required to determine if there was a significant enough back pressure in the third line to halt flow from the rainwater catchment. This required the determination of the pressures of both lines at the proposed point of intersection. The pressure of the rainwater catchment was determined by using a revised version of Bernoulli's Equation seen in equation 3. 6, where point 2 is the intersection of the rainwater catchment to the proposed third line. If the pressure of the third line exceeded that of the pressure in the rainwater catchment connection, then that was an indication that there was no flow from the rainwater catchment. If that was the case, the flow in the third line from Paraíso was eliminated, assuming a valve mechanism at the beginning of the line, causing any flow from Paraíso to travel through the existing two lines. This was also done if the flows from the rainwater catchments were significant enough to warrant complete dedication of the third line during the wet season.

$$p_{RW} = \left(z_1 - z_2 - \frac{{v_2}^2}{2g} - H_f\right) \times \gamma$$

3. 6

Once the head loss was calculated, the pump head for each line was determined using the adjusted Bernoulli's Equation, shown in 3. 7.

$$H_p = \frac{p_2}{\gamma} + z_2 + \frac{{v_2}^2}{2g} + H_f - \frac{p_1}{\gamma} - z_1 - \frac{{v_1}^2}{2g}$$

3. 7

Considering the characteristics of the system that we are analyzing, the pressure and velocity at points 1 and 2 are equal. Thus, pump head simplifies to equation 3. 8.

$$H_p = z_2 + H_f - z_1$$

3. 8

Equation 3. 9 was used to determine the energy needed to be added to the fluid by the pump to move it from the Paraíso Pumping Station to the Miraflores Plant, otherwise known as power or energy out of the pump.

$$P_{out} = Q\gamma H_p$$

3. 9

All the lines require different flows because they have different physical characteristics, including flow, material, and length, but the same pump heads assuming no additional outside influence. The pump heads for the three lines were summed for each alternative, yielding the total power out. For the current operations, the power used by the pumps, or energy in, was determined from the SCADA data. The calculated power needed by the pumps was divided by the known power in to yield the efficiency. This efficiency was used for the remaining alternatives to determine the amount of energy needed to power the pumps by dividing calculated energy out by the efficiency. The total energy in was multiplied by 24 hours and 365 days to estimate annual energy use. This value was then multiplied by $0.14/kWh, which is the average cost of energy per kilowatt hour, to give the annual operating cost of the Pumping Station.

3.3 Design of Pipelines

3.3.1 Design of Third Pipeline

The next phase of the project was to design the third pipeline that will supplement the flow of raw water from the Paraíso Pump Station to the Miraflores Potable Water Plant. A third pipeline will most likely follow a similar path to one of the two current pipelines, because the paths have already been determined and no additional area needs to be acquired, reducing costs. The paths of the current lines can be followed and used as examples of how to overcome any stream, railway, or street crossings that might be encountered. The length of the third pipeline was needed in order to determine construction cost and calculating flow within the pipeline. Another consideration in the design of the third pipeline was the number of stream, railway, and street crossings that the pipeline might encounter, as overcoming these obstacles will increase the construction cost. The last major design consideration was the feasibility of joining the third line with a rainwater catchment line from the Pedro Miguel sub-basin. If it is determined that a rainwater catchment connection is recommended, the junction between the third line and the rainwater catchment line would help determine the path of the third line.

To determine the lengths of the two current pipelines, the Panama Canal Authority internal GIS program was used. On the map of ACP Infrastructure, the lengths of the North pipeline and the South pipeline were determined using a ruler tool that measured out segments of the pipe and summed the total length. Upon closer inspection of the water mains in the GIS program, an approximation on the total number of bends that each pipe made could be determined, allowing for calculation of the minor losses within each pipeline. With the length, number of crossings, and minor losses estimated, a preliminary design was made for the third pipeline.

As a basis to begin the design of the third line, the Engineering & Piping Design Guide was used. This guide was provided by the ACP as supplementary information about the fiberglass reinforced piping system that was preselected as the material for all future pipelines. All of these calculations were conducted for both the North and the South pipeline, which would establish comparisons for the third pipeline. The first step was to determine the minimum pipe diameter for the pipeline using equation 3. 10. This is an equation that is specific to the smooth interior surface of fiberglass piping.

$$D = \frac{0.73\sqrt{\frac{Q}{SG}}}{\rho^{0.33}}$$

3. 10

Where *D* is the minimum diameter, *Q* is the flow within the pipe, *SG* is the specific gravity of the fluid, and ρ is the density of the fluid. The flow values used in this equation were taken from the model, and the minimum diameter was calculated for all of the alternatives.

The next step was to calculate the head loss in the pipeline due to friction. The most commonly used pipe head loss equation is the Darcy-Weisbach Equation. As shown in equation 3. 11, the Darcy-Weisbach Equation calculates the head loss based on a friction factor (related to the pipe roughness), pipe characteristics (length and diameter), fluid characteristics (velocity), and gravity.

$$H_f = f\left(\frac{L}{D}\right)\left(\frac{v^2}{2g}\right)$$

3. 11

Several steps were taken to calculate the friction factor, which is dependent on the flow conditions, pipe diameter and pipe smoothness. First, the Reynolds Number was calculated, which determines the flow conditions within the pipeline. A Reynolds Number of less than 2,000 would indicate laminar flow. A Reynolds Number above 4,000 would indicate turbulent flow. When the type of flow is known, an appropriate equation can be selected to calculate the friction factor. Equation 3. 12 calculates the Reynolds Number.

$$Re = \frac{Dv}{\upsilon}$$

3. 12

Where *Re* is the Reynolds Number, *D* is the pipe diameter, *v* is the velocity of the fluid, and υ is the fluid viscosity. If the Reynolds Number indicated a laminar flow, then equation 3. 13 would be used to calculate the friction factor.

$$f = \frac{64}{Re}$$

3. 13

If the Reynolds Number indicated a turbulent flow, then the Colebrook Equation (equation 3. 14) would be used to calculate the friction factor.

$$\frac{1}{\sqrt{f}} = -2\log\left(\frac{\varepsilon}{3.7D} + \frac{2.51}{Re\sqrt{f}}\right)$$

3. 14

Where ε is the absolute roughness of the pipe. The absolute roughness of a pipe is dependent on the material of the pipe. The constant for fiberglass was found in the Guide. The constant for cast iron was determined from a table (Chaurette, 2003). The Colebrook Equation is an implicit formula, which means that the function in defined by implying a relationship between its argument and its value. In other words, the variable that is being solved for can be found on both sides of the equation, often making it more difficult to solve for the variable. A more direct equation to solve for the friction factor is the Swamee-Jain Equation (equation 3. 15).

$$f = \frac{0.25}{\left(\log_{10}\left(\frac{\varepsilon}{3.7D} + \frac{5.74}{Re^{0.9}}\right)\right)^2}$$

3. 15

Once the friction factor was calculated, it was plugged into equation 3. 11 to solve for head loss due to friction. Friction head loss is considered a major head loss. There are also minor losses in a pipeline.

Minor head losses are due to pressure drops at elbows (bends), tees (junctions), and valves in the pipe. Equation 3. 16 was used to calculate the minor losses in the pipe.

$$H_f = \frac{kv^2}{2g} \tag{3. 16}$$

Where H_f is the minor head loss and k is the flow resistance coefficient. Some of the flow resistance coefficients, such as those for pipe bends, were provided in the fiberglass pipe packet. The other coefficients, such as those for valves and fittings, were obtained through various tables.

These calculations for the North pipeline and the South pipeline helped to determine the best design, including length and pipe diameter, for the third pipeline. A design was chosen with a pipeline path that would follow one of the existing and the most direct path to the Miraflores Potable Water Plant and minimize bends in the pipe. These two considerations would provide the least amount of head loss. Once this design was selected, these parameters were run through the calculations to ensure that the third pipe would not have a large head loss and would be beneficial to add to the current system. While performing these calculations, adjustments were made to the diameter of the third pipeline to determine what size diameter would be appropriate for both the current flow demand and projected future demands.

From this design, a consideration of connecting a rainwater catchment pipeline was also made. A preliminary design was made for the rainwater catchment pipeline and then the two designs were put into the model.

3.3.2 Design of the Rainwater Catchment Pipeline

It is expected that a minimum of a 36 inch diameter pipeline would be necessary to carry the peak flow of 26,000 gpm of water expected during the wet season from the Pedro Miguel sub-basin for rainwater catchment. When considering this option, the potential effect to the flow of water in the third raw water pipeline from Paraíso to Miraflores was considered. The additional flow brought to the pipe from the rainwater catchment could change pressures and change flows so the third pipeline needed to be evaluated to determine if it could handle the potential changes. At this point, changes were made to the third pipeline design to determine if different pipe diameters would do a better job of handling the change of flow and pressure that would come with the rainwater catchment line.

To determine how the rainwater catchment pipeline would join with the third pipeline, the junction loss equation (equation 3. 17) was used to calculate the head loss that would occur at the junction of the third raw water line and the rainwater catchment line. In this preliminary design, the angle is measured from the centerline of the third pipe after the junction, to the centerline of the rainwater catchment pipe.

$$H_j = \frac{Q_0 v_0 - Q_i v_i - Q_1 v_1 \cos\theta}{0.5g(A_0 + A_i)}$$

3. 17

Where H_j is the head loss at the junction, Q is the flow in the pipe, v is the velocity of the fluid, A is the cross-sectional area, θ is the angle between the centerline of the two pipes, and g is the gravitational constant. The subscript 0 denotes parameters of the outlet pipe (third line after juncture), subscript i denotes parameters of the inlet pipe (third line prior to juncture), and subscript 1 denotes the parameters in the lateral pipe (the rainwater catchment pipeline). The location of the inlet pipe, outlet pipe, lateral pipe and θ between the outlet and the lateral pipe can be seen on Figure 22.

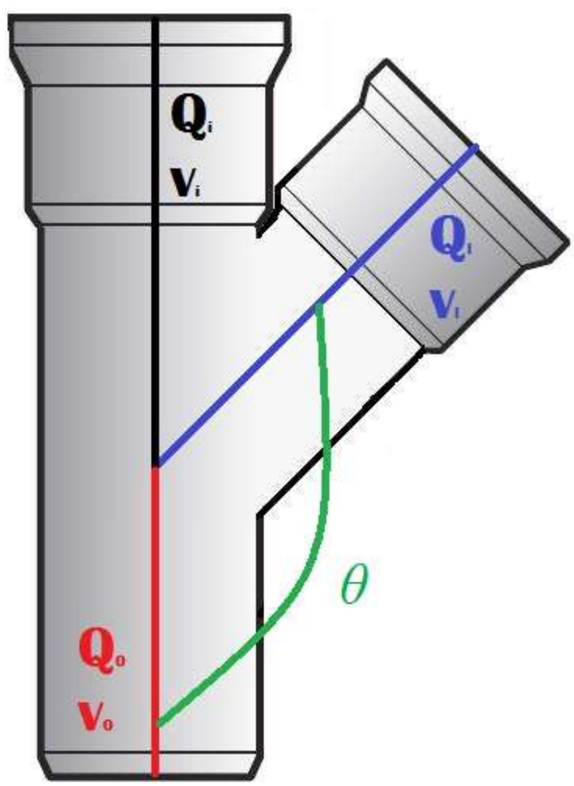


Figure 22 - Pipe Junction Diagram

Different values for θ were inputted into the equation. The smallest value used was 90, which would put the junction as a t-branch. The value for θ was then increased by 5 up to 175.

An approximate location was chosen for where the rainwater catchment would connect with the third line. This location was determined based on the delta of the Pedro Miguel River, as well as considerations about railroad crossings that would occur. Using the GIS ruler tool, the length of the third line prior to the juncture was measured, as well as the length from the juncture to the Miraflores Potable Water Plant. From previous research, it has been determined that approximately 3.5 kilometers of pipeline will be needed from the rainwater catchment to the third line connection. Other considerations that were made in the design of the rainwater catchment included whether any bends would have to occur in the pipeline as it navigated from the upper elevations in the basin down to the elevation of the third pipeline As well as the angle of attachment also had to be considered.

Additional considerations for the rainwater catchment alternatives will include the installation of valves in the third pipeline and in the rainwater catchment pipeline that will be used to help control the flow so that when the rainwater catchment line has a full flow capacity during rainy season, reducing the need of the Paraíso Pumping Station to provide water to the third line. This will also work in the opposite, when the rainwater catchment line is yielding a small flow, the Paraíso Pumping Station can provide a greater flow to meet the demands at the Miraflores Potable Water Plant.

3.4 Projecting Future Needs

It had become apparent that the output for the Miraflores Potable Water Plant will increase in future years due to increased consumption demands from the current 50 to 70 MGD, a 40% increase. Given the current setup of the system, deficiencies with the Pumping Station, and significant pipeline losses, it was necessary to determine how to create that increased capacity. Considering that the energy required for the current Pumping Station to produce the 50 MGD flow is considered inefficient, a third line becomes imperative in the projected scenario. The same method described above was used to determine the energy needed by the pumps for the current demand and the projected demand. Considerations were also made about increasing the diameter size of a third pipeline to increase the water capacity.

3.5 Comparing Alternatives

3.5.1 Determining Costs

The cost of operations for the current system was evaluated on the electricity expended to operate the pumps at the Paraíso Pumping Station at a rate of $0.14/kWh. Other overhead costs, such as payroll and monitoring, remain the same regardless of any proposed improvements to the system, so they were not factored into this analysis. All operations costs were calculated using the model explained above.

After the energy demand and associated energy costs were calculated, the construction cost for a third line and a rainwater catchment was determined from a myriad of sources. The costs of 30” and 36” fiberglass pipe and couplings can be obtained from an ACP file provided by a piping manufacturer. Additional costs for the rainwater catchment system, including an overflow dam and an installation and access road, were determined from an ACP report that previously evaluated the rainwater catchment, altered to fit the proposal parameters.

A rough estimate for pipeline installation costs in Panama was given as 30% to 60% of the procurement cost of the project depending on the terrain, features that the pipeline crosses, and difficulty of installation. When more features are crossed, including roads, waterways, railways, homes, etc., the cost of installation is increased as additional actions need to be taken. For the third pipeline, a conservative assumption for the installation cost was made at 60% due to the necessary crossings and the highly developed surrounding neighborhoods. For the rainwater catchment line, a 30% installation cost was used because the path is through largely undisturbed areas in the rainforest. There was also a 25% contingency added to each of the procurement and installation costs per the ACP standard estimating procedures. Some potential items that would be needed under the installation costs include surveying, excavation, fill, grubbing, and employee wages. A contingency is extra money set aside in case costs rise to a level above just the estimated budget to provide for delays or work stoppages, unforeseen circumstance, or a myriad of other problems construction projects face.

3.5.2 Finding IRR and ANV

For all ACP feasibility analyses, the ultimate consideration is cost, measured by comparing the actual net value (ANV) and calculating the internal rate of return (IRR) to the Authority. These require the initial, annual operating, scheduled maintenance costs, and projected annual profits of the alternative proposed and the current operation. They are concerned with the cost to operate the current system and the cost to install and operate the system with the proposed improvement projected over a 20 year period. Then, ANV and IRR were determined by inputting the values in an excel spreadsheet.

The spreadsheet simplifies the internal rate of return calculation, which factors the cost and profits of the entire project over 20 year span to determine the rate at which the net present value (NPV) equals zero. The spreadsheet first lists costs and profits of each alternative as seen in Figure 23.

		Alternative A							Status Quo						
Year (n)	Fiscal Year	A Investment (-)	B Expenses (-)	C Income (+)	D Loss (-)	Total Cash Flow	Present Value Factor	Present Value	A Investment (-)	B Expenses (-)	C Income (+)	D Loss (-)	Total Cash Flow	Present Value Factor	Present Value
0	2013	-	-	-	-	-	1.0000	-	-	-	-	-	-	1.0000	-
1	2014	-	-	-	-	-	0.8772	-	-	-	-	-	-	0.8772	-
2	2015	-	-	-	-	-	0.7695	-	-	-	-	-	-	0.7695	-
3	2016	-	-	-	-	-	0.6750	-	-	-	-	-	-	0.6750	-
4	2017	-	-	-	-	-	0.5921	-	-	-	-	-	-	0.5921	-
5	2018	-	-	-	-	-	0.5194	-	-	-	-	-	-	0.5194	-
6	2019	-	-	-	-	-	0.4556	-	-	-	-	-	-	0.4556	-
7	2020	-	-	-	-	-	0.3996	-	-	-	-	-	-	0.3996	-
8	2021	-	-	-	-	-	0.3506	-	-	-	-	-	-	0.3506	-
9	2022	-	-	-	-	-	0.3075	-	-	-	-	-	-	0.3075	-
10	2023	-	-	-	-	-	0.2697	-	-	-	-	-	-	0.2697	-
11	2024	-	-	-	-	-	0.2366	-	-	-	-	-	-	0.2366	-
12	2025	-	-	-	-	-	0.2076	-	-	-	-	-	-	0.2076	-
13	2026	-	-	-	-	-	0.1821	-	-	-	-	-	-	0.1821	-
14	2027	-	-	-	-	-	0.1597	-	-	-	-	-	-	0.1597	-
15	2028	-	-	-	-	-	0.1401	-	-	-	-	-	-	0.1401	-
16	2029	-	-	-	-	-	0.1229	-	-	-	-	-	-	0.1229	-
17	2030	-	-	-	-	-	0.1078	-	-	-	-	-	-	0.1078	-
18	2031	-	-	-	-	-	0.0946	-	-	-	-	-	-	0.0946	-
19	2032	-	-	-	-	-	0.0829	-	-	-	-	-	-	0.0829	-
20	2033	-	-	-	-	-	0.0728	-	-	-	-	-	-	0.0728	-
	Totals	-	-	-	-	-		-	-	-	-	-	-		-

Figure 23 - The spreadsheet used to input all monetary values associated with a project.

For both the alternative being considered and the no action alternative (called status quo), Column A is for the investment needed to fund the project. This is considered to be the material and installation costs for all the alternatives. There are no investment costs associated with the status quo. Column B is for all of the annual expenses including the operating costs. For the alternatives and status quo situations, expenses are defined as the cost of the energy needed to operate the pumps. Column C is for any income expected to be generated. There were none associated with any of the alternatives or the status quo as any income incurred by ACP will be the same since they will be producing the same demand, thus negating each other. Column D is for any losses incurred during the operation of the alternative or the status quo. There are no expected losses so this will remain zero for both columns.

	Alt. A vs. Status Quo	
Year (n)	Difference in Cash Flow	Difference in Present Values
0	-	-
1	-	-
2	-	-
3	-	-
4	-	-
5	-	-
6	-	-
7	-	-
8	-	-
9	-	-
10	-	-
11	-	-
12	-	-
13	-	-
14	-	-
15	-	-
16	-	-
17	-	-
18	-	-
19	-	-
20	-	-
Totals	-	-
	IRR	ANV

Figure 24 - Comparing the alternative and status quo to determine Internal Rate of Return (IRR) and Actual Net Value (ANV)

The total cash flow is the sum of column A through D for that particular year. The Present Value Factor is determined first by choosing a minimum rate of return, *r*, based on ACP protocols. Then, equation 3. 18 is used:

$$PVF = (1 + r)^{-n}$$

3. 18

Where n is the year from the beginning of the analysis. Present Value is determined by multiplying the total cash flow by the present value factor for the given year.

From there, the alternative and status quo are compared in the next part of the spreadsheet shown in Figure 24. The difference in cash flow is calculated by taking the total cash flow value of the alternative and subtracting the total cash flow value of the status quo. Then the difference in present value is calculated in the same way with the cash flows.

Internal Rate of Return is then calculated using the Net Present Value Equation (equation 3. 19):

$$NPV = \sum_{n=0}^{N} \frac{C_n}{(1 + r)^n} = 0$$

3. 19

Where NPV is equal to zero and C_n is the difference in cash flows for year n. After inputting all the values, internal rate of return is determined by solving for r. This can be easily solved in excel by using the function "=IRR(values)." A higher IRR is desired because it indicates a higher internal rate of return, promising that any losses would be regained and profits made more quickly. ANV is determined by summing the all of the difference in present values from year 0 to year 20. Once IRRs and ANVs are calculated for all alternatives, they will be compared to determine which would be the most suitable alternative for the 50 MGD and 70 MGD scenarios.

4.0 Results

4.1 Flow and Energy Determination

4.1.1 Current Operations

The ACP's internal GIS system located on their SharePoint site was used to find relevant geographical information and data about the lines including lengths, locations, and elevations. Figure 25 shows a section of the GIS map depicting the Paraíso Pumping Station and the respective lines and pumps in its proximity. The length of the North line and the South line were measured as 4535 meters and 4750 meters, respectively. It was also determined that at approximately 1/3rd of the way from Paraíso, the lines switch their positions with the North line becoming the southern pipeline and vice versa. The result was that both the North and South Lines lengths were more equal than expected. It also made the choice simpler when choosing a path for a third pipeline.

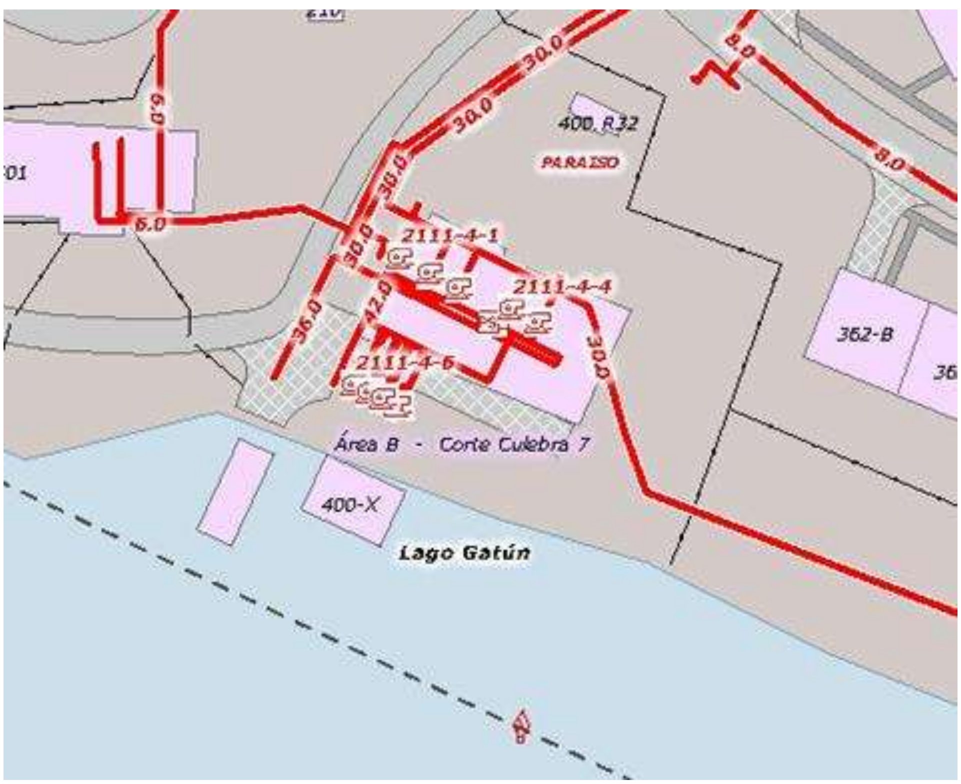


Figure 25 - GIS map of Paraíso Pumping Station (ACP Internal GIS System, 2012).

There were different alternatives considered in order to determine the most energy efficient option for each given demand scenario (50 MGD vs. 70 MGD). For the 50 MGD, the control scenario was the current system and the cost of the pumping energy consumption. An annual value of 11,155,200 kWh for energy used by the Pumping Station was determined from the SCADA data. Then this energy value was multiplied by $0.14/kWh yielded the total cost to operate the Plant of $1,561,728.

The existing flows for the current North and South lines were determined by taking the average of the recorded flows from the SCADA data, yielding values of 13,039 gpm for the North line and 13,252 gpm for the South line. The ratio of flow in the North versus the South line was calculated to be 0.984. After converting all the relevant values to SI units, the friction loss was calculated using the Hazen Williams equation. The friction loss combined with the elevation difference between the Paraíso Pumping Station and the Miraflores Water Treatment Plant gave the pump head of 52.22 m for the North line and 56.19 for the South line. It was expected that pump heads would not quite be equal because of the added flow from Gamboa entering the North line that was not included in the calculations. Pumping energy needed to provide the necessary pump head was then calculated, totaling 881 kWh. Comparing this output energy with the calculated value of known electrical energy used by the Plant yielded an efficiency value of 69.2%.

4.1.2 Alternatives Analysis

The flows from the rainwater catchment would vary monthly based on the climate and precipitation frequency of the watershed during that time. The rainwater catchment flow would originate from a lagoon at the top of the pipeline, artificially created by an overflow dam and flooding along the Pedro Miguel River. These monthly flows were estimated to be constant yearly and are provided in Table 1.

Table 1 - Monthly flow from the Pedro Miguel rainwater catchment and lagoon elevation level.

Month	Outflow (GPM)	Lagoon Elevation (m)
January	10,562	48.5
February	6,000	47.0
March	5,000	43.7

April	4,250	39.9
May	4,200	38.5
June	5,969	38.5
July	11,011	38.5
August	15,400	42.3
September	16,150	38.5
October	19,800	38.5
November	26,000	48.5
December	26,000	48.5

The other alternatives were evaluated in order to determine the necessary energy needed to operate these pumps. A summary of the flows in each line, pump head, annual expended energy and annual operating cost can be seen in Table 2. For complete calculations, please see Appendix A.

Table 2 - Flows, Pump Head, Annual Energy, Annual Operating Cost

	North Line Flow (gpm)	South Line Flow (gpm)	3rd Line Flow (gpm)	Average Pump Head (m)	Annual Energy Used (kWh)	Annual Operating Cost
50 MGD DEMAND						
Current	13,040	13,252	-	54.20	11,155,200	$1,561,728.00
30" Diameter 3rd Line	7,065	7,180	15,120	49.75	3,835,481	$608,518.16
36" Diameter 3rd Line	5,349	5,436	18,558	12.16	2,793,118	$391,036.50
RW Catchment to 30" 3rd Line (low)	1,658	1,685	6,805	3.149	1,489,360	$208,510.33
RW Catchment to 30" 3rd Line (high)	6,038	6,136	12,056	14.72		
70 MGD DEMAND						
30" Diameter 3rd Line	10,382	10,550	22,299	36.66	14,100,306	$1,736,982.18
36" Diameter 3rd Line	7,880	8,009	27,342	22.81	8,978,913	$1,080,696.93
RW Catchment to 30" 3rd Line (low)	11,621	11,810	13,969	44.18	6,919,483	$895,774.78
RW Catchment to 30" 3rd Line (high)	9,373	9,525	20,133	30.69		
RW Catchment to 36" 3rd Line (low)	11,621	11,810	17,128	44.18	4,696,192	$629,531.22
RW Catchment to 36" 3rd Line (high)	7,115	7,231	24,686	19.23		

4.2 Economic Analysis

The pumping costs are not the only associated cost for these alternatives. The procurement and installation costs needed to be considered as well. The procurement cost of a fiberglass line was determined from internal ACP documents. A quote from the fiberglass piping manufacturer lists the price as $144.44 per meter of 30 inch diameter pipe and $186.36 per each coupling. 30 inch diameter fiberglass piping is transported in 11.8 meter sections, necessitating approximately 424 couplings for a 5,000 m long pipe. The procurement costs for the third proposed pipeline then becomes $801,216. Historical records have shown that the installation costs for pipeline projects that cross numerous roads and railways to cost approximately 60% of the procurement cost, a value of $480,730 for this project. There is also a 25% contingency added to the final estimate in order to ensure a sufficient amount of funds for the project, totaling $200,304. Combined, the estimated cost of a 30 inch fiberglass pipeline from Paraíso to Miraflores is approximately $1,482,000. For a third pipeline with a diameter of 36”, the total cost was estimated using the same method and a cost of $272.33 per meter, yielding a total cost of approximately $2,319,000.

For a rainwater catchment line, the cost is significantly higher. With a third pipeline connecting Paraíso and Miraflores, the ACP already owns the path by which the pipeline would follow and the path is close to existing roadways with easy access. With the rainwater catchment line, the path is unclear because most of that land has been largely undisturbed and in a rainforest. As stated above, an analysis for a Pedro Miguel rainwater catchment was conducted prior to the start of this analysis. However, that analysis assumed a direct connection to the Miraflores Potable Water Plant. For this analysis, the rainwater catchment is intended to connect to the proposed third line, resulting in a 3.5 km pipeline to connect to the third line at an elevation of 29 m. The procurement cost of the pipeline includes the fiberglass pipeline at $272.33 per meter of pipeline, an overflow dam to regulate flow from the catchment at $289.00 per cubic meter, and an access roadway for installation and maintenance at $250,000 per kilometer, totaling $2,984,155.00. These values include installation cost. With a 25% contingency cost, the total cost of a rainwater catchment pipeline connected to the 3rd line equals approximately, $3,730,000. See Table 3 for the calculations and a summary of these costs.

Table 3 - Procurement and Installation Cost Estimates for the third proposed line (both 30" and 36") and the rainwater catchment.

3rd Raw Water Line (30")	Unit Price	Quantity	
Fiberglass 30" diameter Pipe Line (m)	$144.44	5000	$722,200.00
Pipe Couplings (ea)	$186.36	424	$79,016.64
60% Installation	$801,216.64	60%	$480,729.98
25% Contingency	$801,216.64	25%	$200,304.16
TOTAL			$1,482,250.78
3rd Raw Water Line (36")			
Fiberglass 36" diameter Pipe Line (m)	$272.33	500	$1,136,650.00
60% Installation	$1,316,650.00	60%	$816,990.00
25% Contingency	$1,316,650.00	25%	$340,412.50
TOTAL			$2,191,575.68
Rainwater Catchment Line (Connect to the 3rd Line)			
Fiberglass 36" diameter Pipe Line (m)	$272.33	3500	$953,155.00
Roller Compacted Concrete Dam (m^3)	$289.00	4000	$1,156,000.00
Roadway (km)	$250,000.00	3.5	$875,000.00
25% Contingency	$2,984,155.00	25%	$746,038.75
TOTAL			$3,730,193.75

Next, the combined installation and operation cost of pipeline was analyzed to determine the Internal Rate of Return and Actual Net Value. The comparisons were broken down into the 50 MGD and 70 MGD demand scenarios. Then the alternatives were compared against the current operations for the 50 MGD scenario and against the operations with a 30 inch diameter third line in operation for the 70 MGD scenario. Table 4 lists the alternatives, their costs, associated IRR and ANV. The alternative letters listed on the left of the description correspond to the tab used to determine these values. These calculations can be seen in Appendix B.

Table 4 - Costs, Internal Rate of Return, and Actual Net Value for Alternatives

	Alternative	**Annual Operating Cost**	**Installation Cost**	**IRR**	**ANV**
	50 MGD DEMAND				
	Current	$1,561,728			
A	30" Diameter 3rd Line	$608,518	$1,482,250	64%	$4,829,843
B	36" Diameter 3rd Line	$391,036	$2,519,052	46%	$5,236,686
C	RW Catchment to 30" 3rd Line	$208,510	$5,212,444	26%	$3,749,096

	70 MGD DEMAND				
	30" Diameter 3rd Line	$1,736,982	$1,482,250		
D	36" Diameter 3rd Line	$1,080,696	$2,519,052	63%	$3,307,774
E	RW Catchment to 30" 3rd Line	$895,774	$5,212,444	22%	$1,840,053
F	RW Catchment to 36" 3rd Line	$629,531	$6,249,246	23%	$2,564,806

4.3 Pipeline Design

4.3.1 Third Pipeline

The lengths of the North and South pipelines were confirmed using the ACP GIS program. It was determined that the North line crosses eleven roads and does not cross the Panama Railway tracks. The South line crosses five roads and crosses under the Panama Railway tracks twice. Minimizing these crossings was the first consideration made for the path of the third line. As the North line leaves the Paraíso Pumping Station, it completes six road crossings (circled in blue in Figure 26) before it joins with the Gamboa line. Between the junction with the Gamboa line and the point where it crosses the South line, the North line does not cross any other roads. It completes the other five road crossings as it approaches the Miraflores Potable Water Plant.

Figure 26 - North line leaving the Paraíso Pumping Station and approaching the Gamboa line (ACP Internal GIS System, 2012)

The South line crosses one road before it crosses the North line. After crossing the North line, it crosses four roads on the way to the Miraflores Potable Water Plant. In order to achieve the shortest distance and minimize the road crossings, it was determined that the third pipeline would initially follow the path of the South line from the Paraíso Pumping Station. The third pipeline will cross one road before the intersection of the North and South line, where the third line would continue to the Miraflores Potable Water Plant following the path of the North line, resulting in a pipe length of 4,320 meters and a total six road crossings. This decision was made to reduce the number of road crossings in order to reduce construction costs (The South line also makes one crossing, while the North line makes six crossings.)

Determining the bends within the pipelines required a bit more opinion than determining the length. Upon closer inspection of the GIS map, it became evident that many of the turns were gradual and took place over a large distance. Bends within the pipelines refer to bends that occur over a relatively small distance. The number of the 45° and the 90° bends in each pipeline were estimated based on sharp angles that could be seen on the GIS map. It was estimated that the North line made seven 90° bends and eleven 45° bends. It was estimated that the South line makes six 90° bends and twelve 45° bends. Based upon the path that was determined above, the third line will make approximately three 90° bends and seven 45° bends.

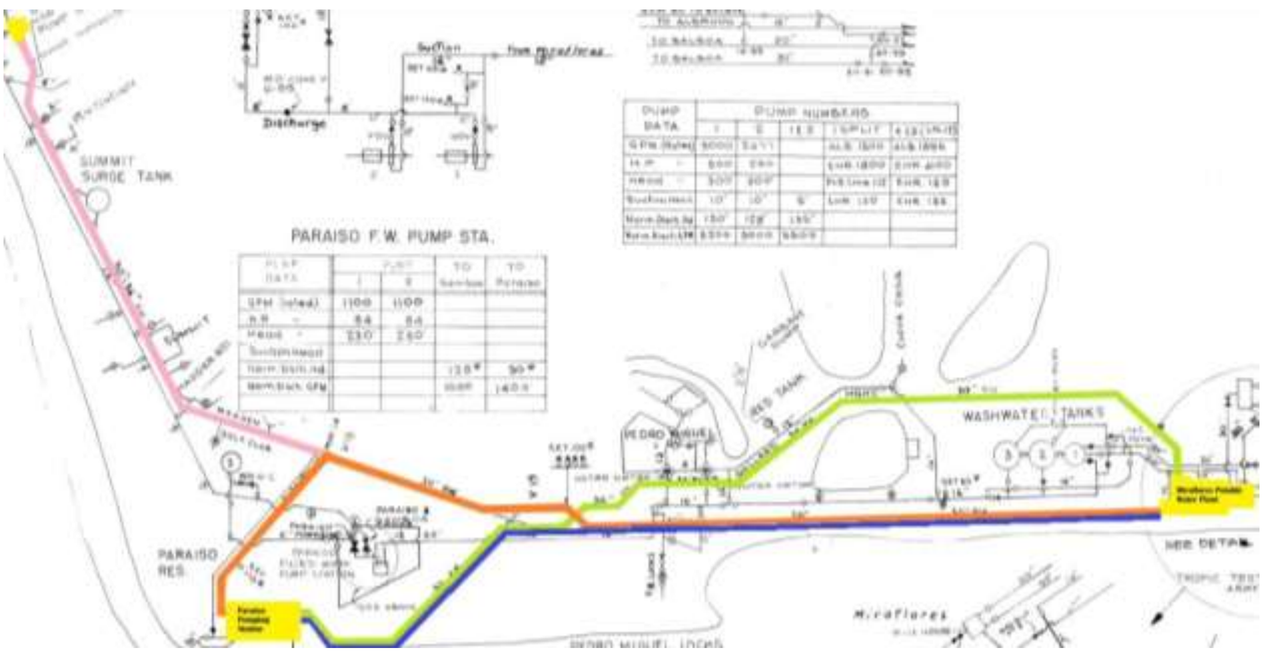


Figure 27 - Approximate Path of the Proposed Third Pipeline (Panama Canal Authority, 1981)

Figure 27 shows an approximate path of the third line, in blue, from the Paraíso Pumping Station to the Miraflores Potable Water Plant. A major contributing factor to this decision was also that the path of the

North line resulted in zero crossings of the Panama Railroad Company railroad tracks, whereas the path of the South line crossed the railroad tracks twice.

The first calculation was to determine the minimum required diameter pipe diameter. For equation 3.10, the specific gravity of water was estimated to be 1.0 and the density of water was estimated to be 62.4 pounds per cubic foot. Based on the calculations, the minimum diameter for the North line ranged between 11.6 and 32.6 inches, depending on the volumetric flow within the line. The diameter of 32.6 inches is the minimum required diameter for the current flow. Based on the calculations, the minimum diameter for the South line ranged between 11.7 and 32.9 inches, depending on the volumetric flow within the line. The diameter of 32.9 inches is the minimum required diameter for the current flow, higher than the current diameter of 30 inches. Based on these calculations, it is apparent that the current pipelines are not large enough to handle current flows without the assistance of the pumps.

The minimum diameter required for the third line ranged between 15.38 and 30.66 inches, depending on the volumetric flow within the pipeline. Based on the calculations, a 30 inch diameter pipe would be sufficient for most of the flows that the pipe would experience; however, as the flow increases, the required diameter approaches 30 inches which indicates that there will be more friction in the line and an increased need for pumping. When flows obtained from the 36" pipeline alternatives are used, a larger pipe calculated minimum diameter (between 30 and 32.9 inches) would be necessary to accommodate the flows. In preparation for anticipated flow increases of up to 70 MGD in the future, a 36 inch diameter may be more beneficial. The benefit of installing a 36 inch pipeline instead of a 30 inch pipeline is a reduction in friction head loss. The reduction in friction head loss reduces the need for pumping, which could result in energy savings.

The various Reynolds Numbers can be seen in Table 5. Although the Reynolds Numbers for each condition are different, all of the values are above 4,000 which indicate that the flows in all of the lines are turbulent, meaning the Colebrook Equation and Swamee-Jain Equation (equation 3. 14 and equation 3. 15, respectively) were used to calculate the friction factor.

Table 5 – Calculated Reynolds Numbers for the three pipelines in all proposed alternatives

Reynolds Numbers		North Line	South Line	Third Line
Alternative		Reynolds Number	Reynolds Number	Reynolds Number
50 MGD Demand				
	Current	980,166	1,004,152	-
A	30" Diameter 3rd Line	538,056	538,056	1,135,532
B	36" Diameter 3rd Line	406,676	406,676	1,165,732
C	Rainwater Catchment to 30" 3rd Line (low)	57,150	57,694	224,790
	Rainwater Catchment to 30" 3rd Line (high)	216,081	219,891	427,264
70 MGD Demand				
	30" Diameter 3rd Line	776,828	788,821	1,661,594
D	36" Diameter 3rd Line	585,483	597,476	1,693,648
E	Rainwater Catchment to 30" 3rd Line (low)	201,386	204,651	398,417
	Rainwater Catchment to 30" 3rd Line (high)	335,824	341,267	663,484
F	Rainwater Catchment to 36" 3rd Line (low)	185,493	188,105	591,747
	Rainwater Catchment to 36" 3rd Line (high)	352,044	357,922	986,246

Based on the friction factor for each alternative, the major head loss was calculated using equation 3.11. The friction factors can be seen in Table 6.

Table 6 – Calculated Friction Factors for the three pipelines in all proposed alternatives

Friction Factor		North Line	South Line	Third Line
Alternative		Friction Factor	Friction Factor	Friction Factor
50 MGD Demand				
	Current	0.014329	0.014307	-
A	30" Diameter 3rd Line	0.015019	0.015019	0.011511
B	36" Diameter 3rd Line	0.015439	0.015439	0.011464
C	Rainwater Catchment to 30" 3rd Line (low)	0.020835	0.015439	0.011464
	Rainwater Catchment to 30" 3rd Line (high)	0.016699	0.016628	0.013547
70 MGD Demand				
	30" Diameter 3rd Line	0.014565	0.014549	0.010863
D	36" Diameter 3rd Line	0.014905	0.014879	0.010832
E	Rainwater Catchment to 30" 3rd Line (low)	0.016831	0.016793	0.013716
	Rainwater Catchment to 30" 3rd Line (high)	0.014905	0.014879	0.012558

F	Rainwater Catchment to 36" 3rd Line (low)	0.017029	0.016994	0.010832
	Rainwater Catchment to 36" 3rd Line (high)	0.014905	0.014879	0.011770

Overall, it seemed that the minor losses were fairly negligible compared to the major losses due to friction. Table 7 shows the major head loss, minor head loss and total head loss for the three lines with each alternative.

Table 7 – Calculated Major, Minor, and Total Head Loss in each pipeline for all proposed alternatives

Total Head Loss		North Line			South Line			Third Line		
	Alternative	Major Head Loss (m)	Minor Head Loss (m)	Total Head Loss (m)	Major Head Loss (m)	Minor Head Loss (m)	Total Head Loss (m)	Major Head Loss (m)	Minor Head Loss (m)	Total Head Loss (m)
50 MGD Demand										
	Current	14.05	0.64	14.69	15.42	0.91	16.33	-	-	-
A	30" Diameter 3rd Line	4.44	0.19	4.63	4.65	0.26	4.91	14.43	0.45	14.88
B	36" Diameter 3rd Line	2.61	0.11	2.72	2.98	0.15	18.20	8.77	0.33	9.10
C	Rainwater Catchment to 30" 3rd Line (low)	2.85	0.64	3.49	0.25	0.02	0.27	8.77	0.09	14.93
	Rainwater Catchment to 30" 3rd Line (high)	3.51	0.14	3.65	2.98	0.91	18.20	10.55	0.45	11.00
70 MGD Demand										
	30" Diameter 3rd Line	8.97	0.40	9.37	9.68	0.56	10.24	29.16	0.97	30.13
D	36" Diameter 3rd Line	5.76	0.64	6.40	5.68	0.91	6.59	17.48	0.70	30.23
E	Rainwater Catchment to 30" 3rd Line (low)	1.66	0.06	1.72	6.28	0.09	10.24	4.98	0.45	5.43
	Rainwater Catchment to 30" 3rd Line (high)	5.76	0.64	6.40	8.19	0.91	9.10	17.48	0.81	30.23
F	Rainwater Catchment to 36" 3rd Line (low)	0.99	0.04	1.03	6.28	0.05	10.24	2.88	0.45	3.33
	Rainwater Catchment to 36" 3rd Line (high)	5.76	0.64	6.40	12.79	0.73	13.52	17.48	0.59	30.23

4.3.2 Rainwater Catchment Pipeline

From previous research completed by the ACP, a 36 inch pipeline would be the minimum pipeline diameter needed to carry the peak flow of approximately 26,000 gpm of water expected during the wet season from the Pedro Miguel sub-basin. A length of 3,500 meters was used in calculations.

When designing the rainwater catchment line, the largest concern was the potential junction between the third raw water line and the rainwater catchment line. Using equation 3. 17 yielded a variety of values for head loss at the junction, including negative numbers. It was determined that a θ value of 165 would be the most efficient junction angle because it yielded the lowest possible head loss values. This angle produced the smallest positive values of head loss for all flow alternatives. The different values can be seen in Table 8.

Table 8 – Calculated Junction Head Loss Values for Different Angles 'θ'

	Maximum Rainwater Catchment Flow (26,000 gpm)			Minimum Rainwater Catchment Flow (4,200 gpm)		
θ	50 MGD (30" pipe)	70 MGD (30" pipe)	70 MGD (36" pipe)	50 MGD (30" pipe)	70 MGD (30" pipe)	70 MGD (36" pipe)
90	0.82959	1.22756	0.87298	0.01484	0.01484	0.01031
95	-0.72352	-0.32565	-0.20564	-0.02419	-0.02419	-0.0168
100	-0.89782	-0.49985	-0.32661	-0.02857	-0.02857	-0.01983
105	0.55656	0.95453	0.68338	0.00798	0.00798	0.00554
110	1.55587	1.95384	1.37734	0.03309	0.03009	0.02298
115	0.66842	1.06639	0.76105	0.01079	0.01079	0.00749
120	-0.83436	-0.4364	-0.28255	-0.02697	-0.02697	-0.01873
125	-0.79948	-0.40151	-0.25832	-0.02609	0.02609	-0.01812
130	0.7231	1.12107	0.79903	0.01217	0.01217	0.845
135	1.55201	1.94997	1.37465	0.03299	0.03299	0.02291
140	0.49969	0.89766	0.64388	0.00655	0.00655	0.00455
145	-0.92622	-0.52825	-0.34634	-0.02928	-0.02928	-0.02033
150	-0.68286	-0.28489	-0.17733	-0.02316	-0.02316	-0.01609
155	0.88112	1.27909	0.90876	0.01613	0.01614	0.01121
160	1.52504	1.923	1.35593	0.03232	0.03232	0.02244
165	0.32637	0.72434	0.52352	0.0022	0.0022	0.00153
170	-0.99758	-0.59961	-0.39589	-0.03107	-0.03107	-0.02158
175	-0.55002	-0.15206	-0.08509	-0.01983	-0.01983	-0.01377

5.0 Discussions and Recommendations

Based on the proposed solutions that were evaluated during this study, it was determined that recommendations would be based on current and future conditions. The first section will discuss the results of the 50 MGD demand of the Miraflores Potable Water Plant and provide recommendations for the ACP to consider in order to immediately reduce the energy consumption of the current system. The second section will discuss the results of the 70 MGD demand of the Miraflores Potable Water Plant and provide recommendations that the ACP may consider when planning for projected future increases of the system.

5.1 Proposed Solution for the 50 MGD Demand Scenario

For the current demand scenario, it is clear that the system is inefficient and other alternatives create a significant energy reduction at the Paraíso Pumping Station while providing a significant return on investment. In order to determine which would be the most suitable for the needs of the ACP, it is necessary to evaluate the alternatives based on their benefits and costs.

For the most basic alternative, a third pipeline, made of fiberglass with a 30 inch diameter, would achieve a 65% energy reduction. The shortest possible length is a result of the shortest path to the Miraflores Plant, following the current South line and then the North line after the point of intersection between the North and South line, helping to reduce pumping and installation costs. The total internal rate of return is 64%, which is very high and significant for such a relatively small investment and the actual net value of $4.8 million.

For a third fiberglass pipeline with a diameter of 36 inches, there is an annual energy reduction of 75%. The increased capacity of this third line would significantly reduce the pumping energy currently wasted on the first two lines. This larger third line would allow a flow almost double the other flows of the other two lines combined. The internal rate of return for this alternative is 46% and the actual net value for the next 20 years is $5.2 million due to the higher procurement and installation cost. While it has a lower rate of return than the 30 inch alternative, this alternative might be more desirable to the ACP decision makers because of the increased rate of return and higher capacity, something that may become very useful for an increased demand scenario.

Finally, the last alternative considered for 50 MGD flow demand involved connecting an approved rainwater catchment from the Pedro Miguel River Basin to the third line, assuming a diameter of 30 inches. A total energy reduction of 86% would be achieved with a rainwater catchment, but the high construction cost of the rainwater catchment yields an IRR value of 26%. The ANV after 20 years is less than that of the 30 inch third pipeline alternative at $3.7 million. While it would conserve the most energy, it is the most expensive option given the current 50 MGD demand.

It is also inevitable that the demand for the Miraflores Potable Water Plant will increase so an infrastructure upgrade made now will assist when the demand increase occurs. According to the CIA, Panama is currently growing at a rate of 1.41% compared to the world growth rate of 1.096%, the growth rate of China at 0.481%, the growth rate of India at 1.312%, and the growth rate of the United States at 0.9% (Advameg, 2012; Central Intelligence Agency, 2012). Much of this growth in Panama is occurring within Panama City, which is the major customer of this water supply system.

5.2 Proposed Solution for the 70 MGD Demand Scenario

For the future demand scenario of approximately 70 MGD, the base comparison case used in the calculations was assuming the most basic of the alternatives was implemented, the 30 inch diameter fiberglass third pipeline. With an increased demand, the system of three pipelines would utilize more energy than it currently does. Increasing the diameter of the third pipeline to 36 inches yields an energy reduction of 37%. When considering connecting a rainwater catchment to this system, a third pipeline with a 30 inch diameter yields an energy reduction of 48%, while a third pipeline with a 36 inch diameter provides an energy reduction of 63%. Looking at the IRR values for each alternative, 63%, 22%, and

23%, respectively, it is clear that an increased pipe diameter size is desirable to achieve energy reduction as well as cost effectiveness.

With this information in mind, it is recommended that for the current demand, the ACP chooses to install the 36 inch third pipeline in order to improve the efficiency of the Paraíso Pumping Station. The larger pipe diameter (36 inches as opposed to 30 inches), will allow for a reduced pressure and friction within the system. The third pipeline should be constructed to have a junction with the rainwater catchment as well as a value at the beginning of the pipeline. When the demand is increased to 70 MGD, this larger pipeline will help the system meet the demands on the Plant. At this point, it is then recommended that the rainwater catchment pipeline be installed to the third pipeline and a butterfly valve be installed at the junction. It is also recommended that any additional demand increase includes an analysis to determine the need to adding another pump at the Paraíso pumping station.

6.0 Conclusion

The goal of this project was to perform a feasibility study for the Panama Canal Authority (ACP) in order to improve upon the current energy of the Paraíso Pumping Station and expected demand increase through analyzing proposed alternatives. Energy usage and flows were determined for the current operations and all proposed alternatives for the two demand scenarios in order to compare against each other. Designs were made as well concerning the location and diameter of the pipeline. In conclusion, two recommendations were made for immediate actions and future actions the ACP should take to improve the energy efficiency of the Paraíso Pumping Station given the two demand scenarios. Considering the age of the pumps and high friction in cast iron pipes, a third fiberglass pipeline with significantly reduced friction and a connection to a gravity flow friction line would greatly aid in the reduction of energy use at the Paraíso Station and increase the fluid outflow of the system.

The purpose of this feasibility analysis was to perform a preliminary evaluation about whether a third pipeline or a rainwater catchment would be economically acceptable given the priorities of the ACP. The design assumed, rather conservatively, installation costs, pumping efficiencies, and pipeline length when performing the economic analysis, indicating that improvements may be at worst what is presented in this report. Additionally, certain assumptions were made about the system that need to be further evaluated if this project were to move forward including constant flow from Gamboa, specific pump efficiencies at the Paraíso Pumping Station, and elevation and area of the basin contributing to the rainwater catchment.

This project made very clear that improvements to the Paraíso Pumping Station would yield significantly positive returns for the ACP because of the currently inefficiency of the station. With the expansion of the Panama Canal concluding in 2014/2015, Panama and its capital Panama City will attract even more global commerce and tourism, adding to the already exponential growth of the area. This growth will eventually become taxing on the current systems' infrastructure, including drink water supply. IDAAN and the ACP should consider investments and improvements in their infrastructure systems now before they are unable to keep up with the demand.

References

Advameg, Inc. (2012). *Republic of Panama*. Encyclopedia of the Nations. Retrieved from http://www.nationsencyclopedia.com/economies/Americas/Panama.html

Bortman, M., Brimblecombe, P., and Cunningham, M.A. (2003). Aeration. In *Environmental Encyclopedia*. (Vol. 1). Detroit: Gale Virtual Reference Library. Retrieved from http://go.galegroup.com.ezproxy.wpi.edu/ps/i.do?id=GALE%7CCX3404800028&v=2.1&u=mlin_c_worpoly&it=r&p=GVRL&sw=w

Bratby, J. (2006). *Coagulation and flocculation in water and wastewater treatment* (2nd ed.). IWA Publishing.

Central Intelligence Agency. (2012). *The World Fact Book.* Retrieved from https://www.cia.gov/library/publications/the-world-factbook/geos/us.html

Chaurette, Jacques. (2003, February). *Pipe Roughness Values*. Retrieved from http://www.pumpfundamentals.com/download-free/pipe_rough_values.pdf

Commonwealth of Australia, 2001. [Untitled Map of Panama showing the Panama Canal]. Retrieved from: http://www.afp.gov.au/media-centre/publications/platypus/previous-editions/2001/june-2001/panama.aspx

Global Security. (2011). *First Isthmian Canal Commission – 1899-1901*. Retrieved from http://www.globalsecurity.org/military/facility/panama-canal-icc.htm

Jones, Garr M. (Eds.). (2008). *Pumping Station Design* (3rd ed). Boston: Elsevier, Inc.

Loucks, D. P., van Beek, E., Stedinger, J. R., Dijkman, J.P.M., Villars, M.T. (2005). *Water Resources Systems Planning and Management: An Introduction to Methods, Models and Applications.* Paris: UNESCO.

Mays, L.W. (2000). *Water Distribution System Handbook*. New York: McGraw-Hill.

Menon, E.S., (2004). *Liquid Pipeline Hydraulics.* New York: Marcel Dekker, Inc.

Panama Canal Authority. (1981). *Section from [Drawing No. 2049-5, Water System Diagram, Pacific Area]*.

Panama Canal Authority. (2000, March). *Panama Canal Customer Newsletter*. Retrieved from http://www.pancanal.com/eng/noticiero/pdf/cust-news-200003.pdf

Panama Canal Authority. (2006, April 24). *Proposal for the Expansion of the Panama Canal*. Retrieved from http://www.pancanal.com/eng/plan/documentos/propuesta/acp-expansion-proposal.pdf

Panama Canal Authority. (2012a). *End of the Construction*. Retrieved from http://www.pancanal.com/eng/history/history/end.html

Panama Canal Authority. (2012b). *The French Canal Construction.* Retrieved from http://www.pancanal.com/eng/history/history/french.html

Panama Canal Authority. (2012c). *American Canal Construction.* Retrieved from http://www.pancanal.com/eng/history/history/american.html

Panama Canal Authority. (2012d). *Canal Frequently Asked Questions – Physical Characteristics*. Retrieved from http://www.pancanal.com/eng/general/canal-faqs/physical.html

Panama Canal Authority. (2012e). *This is the Canal*. Retrieved from http://www.pancanal.com/eng/acp/asi-es-el-canal.html

Panama Canal Authority. (2012f). *Design of the Locks*. Retrieved from http://www.pancanal.com/eng/history/history/locks.html

Panama Canal Authority. (2012g). *Some Early Canal Plans.* Retrieved from http://www.pancanal.com/eng/history/history/early.html

Panama Canal Authority. (2012h). *Work Force*. Retrieved from http://www.pancanal.com/eng/history/history/work.html

Panama Canal Authority. (2012i). *Governors and Administrators – Robert J. Fleming*. Retrieved from http://www.pancanal.com/eng/history/biographies/fleming.html

Panama Canal Authority. (2012j). *Canal Transition – Historic Milestones of the Transition Period*. Retrieved from http://www.pancanal.com/eng/ctransition/milestones.html

Panama Canal Authority. (2012k). *ACP Overview*. Retrieved from http://www.pancanal.com/eng/acp/acp-overview.html

Panama Canal Authority. (2012l). *General Information – Vision*. Retrieved from http://www.pancanal.com/eng/acp/vision-mision.html

Panama Canal Authority. (2012m). *Canal Frequently Asked Questions – Canal Watershed.* Retrieved from http://www.pancanal.com/eng/general/canal-faqs/watershed.html

Panama Canal Authority. (2012, August 15). *Tolls Assessment*. Retrieved from http://www.pancanal.com/eng/op/tolls.html

Panama Canal Company. (1955, May 6). Pure Water Supply Was Major Problem of Primary Concern to Canal Builders. *The Panama Canal Review*, p. 10-11. Retrieved from http://archive.org/stream/panamacanalr51019556pana#page/12/mode/1up

Potter, M. C., Wiggert, D. C., Ramadan, B. H. (2011). *Multimedia Fluid Mechanics* (4th ed.). Cengage Learning

Profile of the Panama Canal System. (2001). Retrieved from http://www.globalsecurity.org/jhtml/jframe.html#http://www.globalsecurity.org/military/facility/images/panama-canal-profile.jpg|||

Raw Water. (2012). Merriam-Webster Dictionary. Retrieved from http://www.merriam-webster.com/dictionary/raw%20water

The U.S. National Archives and Records Administration. (n.d.). *Records of the Panama Canal*. Retrieved from http://www.archives.gov/research/guide-fed-records/groups/185.html#185.6

Water Supply Increase Set. (1964, September 11). *Panama Canal Spillway*, p. 1. Retrieved from http://ufdc.ufl.edu/UF00094771/00475

Winner, D. [Image of Panama Canal Watershed Boundaries]. (n.d.). Panama Guide. Retrieved from http://www.panama-guide.com/article.php/20051027201647939

Winner, D. (2005, October 27). *The Panama Canal Watershed.* Retrieved from http://www.panama-guide.com/article.php/20051027201647939

Appendix A – Flow and Energy Calculations

English		SI Units						
Units	**Constants**							
	Gravity Constant (m/s^2)	9.8						
	kinematic viscosity (m^2/sec)	1.3978						
	Fluid Density (kg/m^3)	0.999244						
				Calculations				
	North Pipeline			**North Pipeline**			Total Pump Head (m)	54.20
	Length (m)	4535		Friction Loss (m)	50.22		Total Energy Out (kWh)	881.0459829
30	Diameter (m)	0.762002		Elevation Difference (m)	2		Calculated Avg Energy In (kWh)	1273.4
	Area (m^2)	0.456038		Pump Head (m)	52.22		Efficiency	69.2%
	Hazen William Constant	69		EnergyOut (kWh)	420.954		Paraíso Flow (MGD)	37860480
13040	Flow (m^3/s)	0.822609					Gamboa Flow (gpm)	5874
							Gamboa Flow (MGD)	8458560
							Total Flow (MGD)	46319040
	South Pipeline			**South Pipeline**				
	Length (m)	4750		Friction Loss (m)	54.19		Annual Energy (kWh)	11,155,200
30	Diameter (m)	0.762002		Elevation Difference (m)	2		Annual Operating Cost	$ 1,561,728.00
	Area (m^2)	0.456038		Pump Head (m)	56.19			
	Hazen William Constant	69		EnergyOut (kWh)	460.3524		North to South flow Ratio	0.984002415
13252	Flow (m^3/s)	0.835983					South to North flow Ratio	1.016257669

Figure 28 - Calculation Table for Current Operations

English Units	Constants	SI Units		Demand (m^3/s)	2.1904		
				Gamboa Flow (m^3/s)	0.3394		
	Gravity Constant (m/s^2)	9.8		Paraíso Flow (m^3/s)	1.8510		
	kinematic viscosity (m^2/sec)	1.3978		South Pipeline Flow (m^3/s)	0.4517		
	Fluid Density (kg/m^3)	0.999244		North Pipeline Flow (m^3/s)	0.4445		
				3rd Pipeline Flow (m^3/s)	0.9548		
				Calculations			
	North Pipeline			**North Pipeline**		Total Pump Head (m)	18.92
	Length (m)	4535		Friction Loss (m)	16.08	Total Energy Out (kWh)	343.294
30	Diameter (m)	0.762002		Elevation Difference (m)	2	Efficiency	69.2%
	Area (m^2)	0.456038		Pump Head (m)	18.08	Total Energy In (kWh)	496.182
	Hazen William Constant	69				Total Paraíso Flow (m^3/s)	1.851
7046.2	Flow (m^3/s)	0.444499				Total Paraíso Flow (gpm)	29342.155
	South Pipeline			**South Pipeline**		Total Paraíso Flow (MGD)	42252702.89
	Length (m)	4750		Friction Loss (m)	17.35	Gamboa Flow (gpm)	5380
30	Diameter (m)	0.762002		Elevation Difference (m)	2	Gamboa Flow (MGD)	7747200
	Area (m^2)	0.456038		Pump Head (m)	19.35	Total Flow (MGD)	49999902.89
	Hazen William Constant	69					
7160.8	Flow (m^3/s)	0.451726				Annual Energy (kWh)	4346558.3
	Third Pipeline			**Third Pipeline**		Annual Operating Cost	$ 608,518.16
	Length (m)	5000		Friction Loss (m)	17.34		
30	Diameter (m)	0.762002		Elevation Difference	2		
	Area (m^2)	0.456038		Pump Head	19.34		
	Hazen William Constant	150					
15135.2	Flow (m^3/s)	0.954782					

Figure 29 - Calculation Table for a 50 MGD Demand on a 30 inch 3rd Fiberglass Pipeline

English				Demand (m^3/s)	2.1904			
Units	**Constants**	SI Units		Gamboa Flow (m^3/s)	0.3394			
	Gravity Constant (m/s^2)	9.8		Paraíso Flow (m^3/s)	1.8510			
	kinematic viscosity (m^2/sec)	1.3978		South Pipeline Flow (m^3/s)	0.3429	5435.7		
	Fluid Density (kg/m^3)	0.999244		North Pipeline Flow (m^3/s)	0.3374	5348.7		
				3rd Pipeline Flow (m^3/s)	1.1707	18557.8		
				Calculations				
	North Pipeline			**North Pipeline**			Total Pump Head (m)	12.16
	Length (m)	4535		Friction Loss (m)	9.66		Total Energy Out (kWh)	220.603
30	Diameter (m)	0.762002		Elevation Difference (m)	2		Efficiency	69.2%
	Area (m^2)	0.456038		Pump Head (m)	11.66		Total Energy In (kWh)	318.849
	Hazen William Constant	69					Total Paraíso Flow (m^3/s)	1.851
5348.7	Flow (m^3/s)	0.337415					Total Paraíso Flow (gpm)	29342.157
	South Pipeline			**South Pipeline**			Total Paraíso Flow (MGD)	42252706.65
	Length (m)	4750		Friction Loss (m)	10.42		Gamboa Flow (gpm)	5380
30	Diameter (m)	0.762002		Elevation Difference (m)	2		Gamboa Flow (MGD)	7747200
	Area (m^2)	0.456038		Pump Head (m)	12.42		Total Flow (MGD)	49999906.65
	Hazen William Constant	69						
5435.7	Flow (m^3/s)	0.3429					Annual Energy (kWh)	2793117.874
	Third Pipeline			**Third Pipeline**			Annual Operating Cost	$ 391,036.50
	Length (m)	5000		Friction Loss (m)	10.41			
36	Diameter (m)	0.914402		Elevation Difference	2			
	Area (m^2)	0.656695		Pump Head	12.41			
	Hazen William Constant	150						
18557.8	Flow (m^3/s)	1.170691						

Figure 30 - Calculation Table for a 50 MGD Demand on a 36 inch 3rd Fiberglass Pipeline

Constants													
Gravity Constant (m/s^2)	9.8												
kinematic viscosity (m^2/sec)	1.3978												
Fluid Density (kg/m^3)	0.999244												
Demand (m^3/s)		2.1904	2.1904	2.1904	2.1904	2.1904	2.1904	2.1904	2.1904	2.1904	2.1904	2.1904	2.1904
Gamboa Flow (m^3/s)		0.3394	0.3394	0.3394	0.3394	0.3394	0.3394	0.3394	0.3394	0.3394	0.3394	0.3394	0.3394
Paraíso Flow (m^3/s)		1.1847	1.4725	1.5356	1.5829	1.5861	1.4745	1.1564	0.8795	0.8322	0.6020	0.2108	0.2108
South Pipeline Flow (m^3/s)		0.2891	0.3594	0.3748	0.3863	0.3871	0.3598	0.2822	0.2146	0.2031	0.3034	0.1063	0.1063
North Pipeline Flow (m^3/s)		0.2845	0.3536	0.3688	0.3801	0.3809	0.3541	0.2777	0.2112	0.1998	0.2986	0.1046	0.1046
3rd Pipeline Flow (m^3/s)		0.6111	0.7595	0.7921	0.8165	0.8181	0.7606	0.5965	0.4537	0.4293			
North Pipeline		**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sept**	**Oct**	**Nov**	**Dec**
Length (m)		4535	4535	4535	4535	4535	4535	4535	4535	4535	4535	4535	4535
Diameter (in)		30	30	30	30	30	30	30	30	30	30	30	30
Diameter (m)		0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762
Area (m^2)		0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456
Hazen William Constant		69	69	69	69	69	69	69	69	69	69	69	69
Flow (gpm)		4510	5605	5846	6026	6038	5613	4402	3348	3168	4733	1658	1658
Flow (m^3/s)		0.284	0.354	0.369	0.380	0.381	0.354	0.278	0.211	0.200	0.299	0.105	0.105
Friction Loss (m)		7.04	10.53	11.38	12.04	12.08	10.56	6.74	4.06	3.66	7.70	1.11	1.11
Elevation Difference (m)		2	2	2	2	2	2	2	2	2	2	2	2
Pump Head (m)		9.04	12.53	13.38	14.04	14.08	12.56	8.74	6.06	5.66	9.70	3.11	3.11
Energy Out (kWh)		25.214	43.428	48.360	52.298	52.568	43.576	23.772	12.542	11.094	28.383	3.183	3.183
South Pipeline													
Length (m)		4750	4750	4750	4750	4750	4750	4750	4750	4750	4750	4750	4750
Diameter (in)		30	30	30	30	30	30	30	30	30	30	30	30
Diameter (m)		0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762
Area (m^2)		0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456
Hazen William Constant		69	69	69	69	69	69	69	69	69	69	69	69
Flow (gpm)		4583	5697	5941	6124	6136	5704	4474	3403	3219	4810	1685	1685
Flow (m^3/s)		0.289	0.359	0.375	0.386	0.387	0.360	0.282	0.215	0.203	0.303	0.106	0.106
Friction Loss (m)		7.60	11.37	12.28	12.99	13.04	11.39	7.27	4.38	3.95	8.31	1.19	1.19
Elevation Difference (m)		2	2	2	2	2	2	2	2	2	2	2	2
Pump Head (m)		9.60	13.37	14.28	14.99	15.04	13.39	9.27	6.38	5.95	10.31	3.19	3.19
Energy Out (kWh)		27.204	47.069	52.454	56.755	57.050	47.230	25.633	13.422	11.852	30.656	3.326	3.326
Third Pipeline													
Length (m)		5000	5000	5000	5000	5000	5000	5000	5000	5000	5000	5000	5000
Diameter (in)		30	30	30	30	30	30	30	30	30	30	30	30
Diameter (m)		0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762
Area (m^2)		0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456
Hazen William Constant		150	150	150	150	150	150	150	150	150	150	150	150
Flow (gpm)		9687	12040	12556	12943	12969	12056	9456	7192	6805	0	0	0
Flow (m^3/s)		0.611	0.760	0.792	0.816	0.818	0.761	0.596	0.454	0.429	0.000	0.000	0.000
Friction Loss (m)		7.60	11.36	12.27	12.98	13.03	11.38	7.26	4.38	3.95	0.00	0.00	0.00
Elevation Difference		2	2	2	2	2	2	2	2	2	2	2	2
Pump Head		9.60	13.36	14.27	14.98	15.03	13.38	9.26	6.38	5.95	2.00	2.00	2.00
Energy Out (kWh)		57.464	99.424	110.797	119.882	120.505	99.764	54.147	28.354	25.038	0.000	0.000	0.000
Rainwater Catchment													
Length (m)		3500	3500	3500	3500	3500	3500	3500	3500	3500	3500	3500	3500
Diameter (in)		36	36	36	36	36	36	36	36	36	36	36	36
Diameter (m)		0.9144	0.9144	0.9144	0.9144	0.9144	0.9144	0.9144	0.9144	0.9144	0.9144	0.9144	0.9144
Area (m^2)		0.6567	0.6567	0.6567	0.6567	0.6567	0.6567	0.6567	0.6567	0.6567	0.6567	0.6567	0.6567
Hazen William Constant		150	150	150	150	150	150	150	150	150	150	150	150
Flow (gpm)		10562	6000	5000	4250	4200	5969	11011	15400	16150	19800	26000	26000
Flow (m^3/s)		0.666	0.379	0.315	0.268	0.265	0.377	0.695	0.971	1.019	1.249	1.640	1.640
Friction Loss (m)		2.567	0.902	0.644	0.477	0.466	0.893	2.773	5.158	5.632	8.211	13.592	13.592
Velocity (m/s)		1.015	0.576	0.480	0.408	0.403	0.573	1.058	1.479	1.551	1.902	2.498	2.498
Lagoon Elevation (m)		48.5	47	43.7	39.9	38.5	38.5	38.5	42.3	38.5	38.5	48.5	48.5
Elevation Difference (m)		19.5	18	14.7	10.9	9.5	9.5	9.5	13.3	9.5	9.5	19.5	19.5
Gauge Pressure (kPa)		165.432	167.273	137.512	101.946	88.329	84.059	65.396	79.052	37.127	11.680	56.653	56.653
Gauge Pressure (psi)		23.993	24.260	19.944	14.785	12.811	12.191	9.485	11.465	5.385	1.694	8.217	8.217
Total Pump Head (m)		9.41	13.08	13.98	14.67	14.72	13.11	9.09	6.27	5.86	10.01	3.15	3.15
Total Energy Out (kWh)		109.29	188.82	210.37	227.58	228.76	189.47	103.00	54.06	47.77	59.02	6.51	6.51
Efficiency		69.2%	69.2%	69.2%	69.2%	69.2%	69.2%	69.2%	69.2%	69.2%	69.2%	69.2%	69.2%
Total Energy In (kWh)		157.96	272.91	304.06	328.94	330.65	273.85	148.87	78.14	69.04	85.31	9.41	9.41
Total Paraíso Flow (m^3/s)		1.18	1.47	1.54	1.58	1.59	1.47	1.16	0.88	0.83	0.60	0.21	0.21
Total Paraíso Flow (gpm)		18,780	23,342	24,342	25,092	25,142	23,373	18,331	13,942	13,192	9,542	3,342	3,342
Total Paraíso Flow (MGD)		27,043,536	33,612,820	35,052,821	36,132,821	36,204,821	33,657,460	26,396,976	20,076,812	18,996,811	13,740,817	4,812,806	4,812,806
Gamboa Flow (gpm)		5,380	5,380	5,380	5,380	5,380	5,380	5,380	5,380	5,380	5,380	5,380	5,380
Gamboa Flow (MGD)		7,747,200	7,747,200	7,747,200	7,747,200	7,747,200	7,747,200	7,747,200	7,747,200	7,747,200	7,747,200	7,747,200	7,747,200
RW Catchment Flow (MGD)		15,209,280	8,640,000	7,200,000	6,120,000	6,048,000	8,595,360	15,855,840	22,176,000	23,256,000	28,512,000	37,440,000	37,440,000
Total Flow (MGD)		50,000,016	50,000,020	50,000,021	50,000,021	50,000,021	50,000,020	50,000,016	50,000,012	50,000,011	50,000,017	50,000,006	50,000,006
Monthy Energy (kWh)		113734.61	196498.71	218925.26	236837.46	238065.59	197169.44	107187.80	56262.73	49710.21	61423.53	6772.09	6772.09
Monthy Operating Cost		$ 15,922.85	$ 27,509.82	$ 30,649.54	$ 33,157.24	$ 33,329.18	$ 27,603.72	$ 15,006.29	$ 7,876.78	$ 6,959.43	$ 8,599.29	$ 948.09	$ 948.09
Total Annual Cost	$ 208,510.33												

Figure 31 - Calculation Table for a 50 MGD Demand on a 30 inch 3rd Fiberglass Pipeline with a connection to the Pedro Miguel Rainwater Catchment

English Units	**Constants**	SI Units		Demand (m^3/s)	3.0666			
				Gamboa Flow (m^3/s)	0.3394			
	Gravity Constant (m/s^2)	9.8		Paraíso Flow (m^3/s)	2.7272			
	kinematic viscosity (m^2/sec)	1.3978		South Pipeline Flow (m^3/s)	0.6655	10550.3		
	Fluid Density (kg/m^3)	0.999244		North Pipeline Flow (m^3/s)	0.6549	10381.5		
				3rd Pipeline Flow (m^3/s)	1.4067	22299.3		
				Calculations				
	North Pipeline			**North Pipeline**			Total Pump Head (m)	36.66
	Length (m)	4535		Friction Loss (m)	32.94		Total Energy Out (kWh)	979.915
30	Diameter (m)	0.762002		Elevation Difference (m)	2		Efficiency	69.2%
	Area (m^2)	0.456		Pump Head (m)	34.94		Total Energy In (kWh)	1416.33
	Hazen William Constant	69					Total Paraíso Flow (m^3/s)	2.73
10382	Flow (m^3/s)	0.655					Total Paraíso Flow (gpm)	43231
	South Pipeline			**South Pipeline**			Total Paraíso Flow (MGD)	62252837
	Length (m)	4750		Friction Loss (m)	35.54		Gamboa Flow (gpm)	5380
30	Diameter (m)	0.762002		Elevation Difference (m)	2		Gamboa Flow (MGD)	7747200
	Area (m^2)	0.456		Pump Head (m)	37.54		Total Flow (MGD)	70,000,037
	Hazen William Constant	69						
10550	Flow (m^3/s)	0.666					Annual Energy (kWh)	12407015.57
	Third Pipeline			**Third Pipeline**			Annual Operating Cost	$ 1,736,982.18
	Length (m)	5000		Friction Loss (m)	35.52			
30	Diameter (m)	0.762002		Elevation Difference	2			
	Area (m^2)	0.456		Pump Head	37.52			
	Hazen William Constant	150						
22299	Flow (m^3/s)	1.407						

Figure 32 - Calculation Table for a 70 MGD Demand on a 30 inch 3rd Fiberglass Pipeline

English Units	Constants	SI Units		Demand (m^3/s)	3.0666			
				Gamboa Flow (m^3/s)	0.3394			
	Gravity Constant (m/s^2)	9.8		Paraíso Flow (m^3/s)	2.7272			
	kinematic viscosity (m^2/sec)	1.3978		South Pipeline Flow (m^3/s)	0.5052	8008.6		
	Fluid Density (kg/m^3)	0.999244		North Pipeline Flow (m^3/s)	0.4971	7880.5		
				3rd Pipeline Flow (m^3/s)	1.7248	27342.0		
				Calculations				
	North Pipeline			**North Pipeline**			Total Pump Head (m)	22.81
	Length (m)	4535		Friction Loss (m)	19.78		Total Energy Out (kWh)	609.67
30	Diameter (m)	0.762002		Elevation Difference (m)	2		Efficiency	69.2%
	Area (m^2)	0.456		Pump Head (m)	21.78		Total Energy In (kWh)	881.19
	Hazen William Constant	69					Total Paraíso Flow (m^3/s)	2.73
7880.5	Flow (m^3/s)	0.497					Total Paraíso Flow (gpm)	43231
							Total Paraíso Flow (MGD)	62252828
	South Pipeline			**South Pipeline**			Gamboa Flow (gpm)	5380
	Length (m)	4750		Friction Loss (m)	21.34		Gamboa Flow (MGD)	7747200
30	Diameter (m)	0.762002		Elevation Difference (m)	2		Total Flow (MGD)	70,000,028
	Area (m^2)	0.456		Pump Head (m)	23.34			
	Hazen William Constant	69					Annual Energy (kWh)	7719263.795
8009	Flow (m^3/s)	0.505211					Annual Operating Cost	$ 1,080,696.93
	Third Pipeline			**Third Pipeline**				
	Length (m)	5000		Friction Loss (m)	21.31			
36	Diameter (m)	0.914		Elevation Difference	2			
	Area (m^2)	0.657		Pump Head	23.31			
	Hazen William Constant	150						
27342.04	Flow (m^3/s)	1.725						

Figure 33 - Calculation Table for a 70 MGD Demand on a 36 inch 3rd Fiberglass Pipeline

Demand (m^3/s)		3.0666	3.0666	3.0666	3.0666	3.0666	3.0666	3.0666	3.0666	3.0666	3.0666	3.0666	3.0666
Gamboa Flow (m^3/s)		0.3394	0.3394	0.3394	0.3394	0.3394	0.3394	0.3394	0.3394	0.3394	0.3394	0.3394	0.3394
Paraíso Flow (m^3/s)		2.0609	2.3487	2.4118	2.4591	2.4622	2.3506	2.0326	1.7557	1.7084	1.4781	1.0870	1.0870
South Pipeline Flow (m^3/s)		0.5029	0.5732	0.5886	0.6001	0.6009	0.5737	0.4960	0.4285	0.4169	0.7450	0.5479	0.5479
North Pipeline Flow (m^3/s)		0.4949	0.5640	0.5792	0.5905	0.5913	0.5645	0.4881	0.4216	0.4102	0.7331	0.5391	0.5391
3rd Pipeline Flow (m^3/s)		1.0630	1.2115	1.2440	1.2684	1.2701	1.2125	1.0484	0.9056	0.8812			
North Pipeline		**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sept**	**Oct**	**Nov**	**Dec**
Length (m)		4535	4535	4535	4535	4535	4535	4535	4535	4535	4535	4535	4535
Diameter (in)		30	30	30	30	30	30	30	30	30	30	30	30
Diameter (m)		0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762
Area (m^2)		0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456
Hazen William Constant		69	69	69	69	69	69	69	69	69	69	69	69
Flow (gpm)		7845	8941	9181	9361	9373	8948	7737	6683	6503	11621	8546	8546
Flow (m^3/s)		0.495	0.564	0.579	0.591	0.591	0.564	0.488	0.422	0.410	0.733	0.539	0.539
Friction Loss (m)		19.62	24.98	26.24	27.20	27.26	25.02	19.12	14.58	13.86	40.58	22.98	22.98
Elevation Difference (m)		2	2	2	2	2	2	2	2	2	2	2	2
Pump Head (m)		21.62	26.98	28.24	29.20	29.26	27.02	21.12	16.58	15.86	42.58	24.98	24.98
Energy Out (kWh)		104.836	149.137	160.268	168.969	169.560	149.474	101.024	68.515	63.779	305.902	131.981	131.981
South Pipeline													
Length (m)		4750	4750	4750	4750	4750	4750	4750	4750	4750	4750	4750	4750
Diameter (in)		30	30	30	30	30	30	30	30	30	30	30	30
Diameter (m)		0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762
Area (m^2)		0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456
Hazen William Constant		69	69	69	69	69	69	69	69	69	69	69	69
Flow (gpm)		7973	9086	9330	9513	9525	9094	7863	6792	6609	11810	8685	8685
Flow (m^3/s)		0.503	0.573	0.589	0.600	0.601	0.574	0.496	0.428	0.417	0.745	0.548	0.548
Friction Loss (m)		21.17	26.96	28.31	29.35	29.42	27.00	20.63	15.74	14.96	43.79	24.80	24.80
Elevation Difference (m)		2	2	2	2	2	2	2	2	2	2	2	2
Pump Head (m)		23.17	28.96	30.31	31.35	31.42	29.00	22.63	17.74	16.96	45.79	26.80	26.80
Energy Out (kWh)		114.191	162.665	174.849	184.373	185.020	163.034	110.021	74.474	69.298	334.319	143.891	143.891
Third Pipeline													
Length (m)		5000	5000	5000	5000	5000	5000	5000	5000	5000	5000	5000	5000
Diameter (in)		30	30	30	30	30	30	30	30	30	30	30	30
Diameter (m)		0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762
Area (m^2)		0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456
Hazen William Constant		150	150	150	150	150	150	150	150	150	150	150	150
Flow (gpm)		16851	19204	19720	20107	20133	19220	16620	14356	13969	0	0	0
Flow (m^3/s)		1.063	1.211	1.244	1.268	1.270	1.212	1.048	0.906	0.881	0.000	0.000	0.000
Friction Loss (m)		21.15	26.94	28.29	29.33	29.40	26.98	20.62	15.72	14.95	0.00	0.00	0.00
Elevation Difference		2	2	2	2	2	2	2	2	2			
Pump Head		23.15	28.94	30.29	31.33	31.40	28.98	22.62	17.72	16.95	0.00	0.00	0.00
Energy Out (kWh)		241.195	343.577	369.310	389.425	390.792	344.356	232.386	157.307	146.375	0.000	0.000	0.000
Rainwater Catchment													
Length (m)		3500	3500	3500	3500	3500	3500	3500	3500	3500	3500	3500	3500
Diameter (in)		36	36	36	36	36	36	36	36	36	36	36	36
Diameter (m)		0.914	0.914	0.914	0.914	0.914	0.914	0.914	0.914	0.914	0.914	0.914	0.914
Area (m^2)		0.657	0.657	0.657	0.657	0.657	0.657	0.657	0.657	0.657	0.657	0.657	0.657
Hazen William Constant		150	150	150	150	150	150	150	150	150	150	150	150
Flow (gpm)		10562	6000	5000.00	4250	4200	5969	11011	15400	16150	19800	26000	26000
Flow (m^3/s)		0.666	0.379	0.315	0.268	0.265	0.377	0.695	0.971	1.019	1.249	1.640	1.640
Friction Loss (m)		2.567	0.902	0.644	0.477	0.466	0.893	2.773	5.158	5.632	8.211	13.592	13.592
Velocity (m/s)		1.015	0.576	0.480	0.408	0.403	0.573	1.058	1.479	1.551	1.902	2.498	2.498
Lagoon Elevation (m)		48.5	47	43.7	39.9	38.5	38.5	38.5	42.3	38.5	38.5	48.5	48.5
Elevation Difference (m)		19.5	18	14.7	10.9	9.5	9.5	9.5	13.3	9.5	9.5	19.5	19.5
Gauge Pressure (kPa)		165.432	167.273	137.512	101.946	88.329	84.059	65.396	79.052	37.127	11.680	56.653	56.653
Gauge Pressure (psi)		23.993	24.260	19.944	14.785	12.811	12.191	9.485	11.465	5.385	1.694	8.217	8.217
Total Pump Head (m)		22.65	28.29	29.61	30.63	30.69	28.33	22.12	17.35	16.59	44.18	25.89	25.89
Total Energy Out (kWh)		457.36	651.22	699.94	738.03	740.62	652.70	440.68	298.48	277.78	640.03	275.79	275.79
Efficiency		69.2%	69.2%	69.2%	69.2%	69.2%	69.2%	69.2%	69.2%	69.2%	69.2%	69.2%	69.2%
Total Energy In (kWh)		661.05	941.25	1011.67	1066.71	1070.45	943.38	636.94	431.41	401.48	925.07	398.62	398.62
Total Paraíso Flow (m^3/s)		2.06	2.35	2.41	2.46	2.46	2.35	2.03	1.76	1.71	1.48	1.09	1.09
Total Paraíso Flow (gpm)		32,669.13	37,231.13	38,231.13	38,981.13	39,031.13	37,262.13	32,220.13	27,831.13	27,081.13	23,431.14	17,231.13	17,231.13
Total Paraíso Flow (MGD)		47,043,548	53,612,832	55,052,832	56,132,833	56,204,833	53,657,472	46,396,987	40,076,824	38,996,823	33,740,841	24,812,830	24,812,830
Gamboa Flow (gpm)		5,380	5,380	5,380	5,380	5,380	5,380	5,380	5,380	5,380	5,380	5,380	5,380
Gamboa Flow (MGD)		7,747,200	7,747,200	7,747,200	7,747,200	7,747,200	7,747,200	7,747,200	7,747,200	7,747,200	7,747,200	7,747,200	7,747,200
RW Catchment Flow (MGD)		15209280	8640000	7200000	6120000	6048000	8595360	15855840	22176000	23256000	28512000	37440000	37440000
Total Flow (MGD)		70,000,028	70,000,032	70,000,032	70,000,033	70,000,033	70,000,032	70,000,027	70,000,024	70,000,023	70,000,041	70,000,030	70,000,030
Monthy Energy (kWh)		475952.64	677698.01	728401.50	768034.02	770725.75	679233.56	458593.60	310617.25	289068.45	666053.85	287006.34	287006.34
Monthy Operating Cost		$ 66,633.37	$ 94,877.72	$ 101,976.21	$ 107,524.76	$ 107,901.61	$ 95,092.70	$ 64,203.10	$ 43,486.41	$ 40,469.58	$ 93,247.54	$ 40,180.89	$ 40,180.89
Total Annual Cost	$ 895,775												

Figure 34 - Calculation Table for a 70 MGD Demand on a 30 inch 3rd Fiberglass Pipeline with a connection to the Pedro Miguel Rainwater Catchment

Demand (m^3/s)		3.0666	3.0666	3.0666	3.0666	3.0666	3.0666	3.0666	3.0666	3.0666	3.0666	3.0666	3.0666
Gamboa Flow (m^3/s)		0.3394	0.3394	0.3394	0.3394	0.3394	0.3394	0.3394	0.3394	0.3394	0.3394	0.3394	0.3394
Paraíso Flow (m^3/s)		2.0609	2.3487	2.4118	2.4591	2.4622	2.3506	2.0326	1.7557	1.7084	1.4781	1.0870	1.0870
South Pipeline Flow (m^3/s)		0.3818	0.4351	0.4468	0.4555	0.4561	0.4355	0.3765	0.3252	0.3165	0.7450	0.5479	0.5479
North Pipeline Flow (m^3/s)		0.3757	0.4281	0.4396	0.4483	0.4488	0.4285	0.3705	0.3200	0.3114	0.7331	0.5391	0.5391
3rd Pipeline Flow (m^3/s)		1.3034	1.4854	1.5253	1.5553	1.5573	1.4867	1.2855	1.1104	1.0805			
North Pipeline		**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sept**	**Oct**	**Nov**	**Dec**
Length (m)		4535	4535	4535	4535	4535	4535	4535	4535	4535	4535	4535	4535
Diameter (in)		30	30	30	30	30	30	30	30	30	30	30	30
Diameter (m)		0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762
Area (m^2)		0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456
Hazen William Constant		69	69	69	69	69	69	69	69	69	69	69	69
Flow (gpm)		5955	6787	6969	7106	7115	6792	5873	5073	4937	11621	8546	8546
Flow (m^3/s)		0.376	0.428	0.440	0.448	0.449	0.428	0.371	0.320	0.311	0.733	0.539	0.539
Friction Loss (m)		11.78	15.00	15.76	16.33	16.37	15.03	11.48	8.76	8.33	40.58	22.98	22.98
Elevation Difference (m)		2	2	2	2	2	2	2	2	2	2	2	2
Pump Head (m)		13.78	17.00	17.76	18.33	18.37	17.03	13.48	10.76	10.33	42.58	24.98	24.98
Energy Out (kWh)		50.732	71.338	76.502	80.536	80.810	71.494	48.954	33.739	31.513	305.902	131.981	131.981
South Pipeline													
Length (m)		4750	4750	4750	4750	4750	4750	4750	4750	4750	4750	4750	4750
Diameter (in)		30	30	30	30	30	30	30	30	30	30	30	30
Diameter (m)		0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762
Area (m^2)		0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456	0.456
Hazen William Constant		69	69	69	69	69	69	69	69	69	69	69	69
Flow (gpm)		6052	6897	7082	7221	7231	6903	5969	5156	5017	11810	8685	8685
Flow (m^3/s)		0.382	0.435	0.447	0.456	0.456	0.435	0.377	0.325	0.316	0.745	0.548	0.548
Friction Loss (m)		12.71	16.19	17.00	17.63	17.67	16.21	12.39	9.45	8.98	43.79	24.80	24.80
Elevation Difference (m)		2	2	2	2	2	2	2	2	2	2	2	2
Pump Head (m)		14.71	18.19	19.00	19.63	19.67	18.21	14.39	11.45	10.98	45.79	26.80	26.80
Energy Out (kWh)		55.045	77.559	83.205	87.615	87.915	77.730	53.102	36.496	34.068	334.319	143.891	143.891
Third Pipeline													
Length (m)		5000	5000	5000	5000	5000	5000	5000	5000	5000	5000	5000	5000
Diameter (in)		36	36	36	36	36	36	36	36	36	36	36	36
Diameter (m)		0.914	0.914	0.914	0.914	0.914	0.914	0.914	0.914	0.914	0.914	0.914	0.914
Area (m^2)		0.657	0.657	0.657	0.657	0.657	0.657	0.657	0.657	0.657	0.657	0.657	0.657
Hazen William Constant		150	150	150	150	150	150	150	150	150	150	150	150
Flow (gpm)		20662	23547	24180	24654	24686	23567	20378	17602	17128	0	0	0
Flow (m^3/s)		1.303	1.485	1.525	1.555	1.557	1.487	1.286	1.110	1.080	0.000	0.000	0.000
Friction Loss (m)		12.69	16.16	16.98	17.60	17.64	16.19	12.37	9.44	8.97	0.00	0.00	0.00
Elevation Difference		2	2	2	2	2	2	2	2	2			
Pump Head		14.69	18.16	18.98	19.60	19.64	18.19	14.37	11.44	10.97	0.00	0.00	0.00
Energy Out (kWh)		187.674	264.427	283.673	298.707	299.728	265.010	181.053	124.441	116.164	0.000	0.000	0.000
Rainwater Catchment													
Length (m)		3500	3500	3500	3500	3500	3500	3500	3500	3500	3500	3500	3500
Diameter (in)		36	36	36	36	36	36	36	36	36	36	36	36
Diameter (m)		0.9144	0.9144	0.9144	0.9144	0.9144	0.9144	0.9144	0.9144	0.9144	0.9144	0.9144	0.9144
Area (m^2)		0.6567	0.6567	0.6567	0.6567	0.6567	0.6567	0.6567	0.6567	0.6567	0.6567	0.6567	0.6567
Hazen William Constant		150	150	150	150	150	150	150	150	150	150	150	150
Flow (gpm)		10562	6000	5000	4250	4200	5969	11011	15400	16150	19800	26000	26000
Flow (m^3/s)		0.666	0.379	0.315	0.268	0.265	0.377	0.695	0.971	1.019	1.249	1.640	1.640
Friction Loss (m)		2.567	0.902	0.644	0.477	0.466	0.893	2.773	5.158	5.632	8.211	13.592	13.592
Velocity (m/s)		1.015	0.576	0.480	0.408	0.403	0.573	1.058	1.479	1.551	1.902	2.498	2.498
Lagoon Elevation (m)		48.5	47	43.7	39.9	38.5	38.5	38.5	42.3	38.5	38.5	48.5	48.5
Elevation Difference (m)		19.5	18	14.7	10.9	9.5	9.5	9.5	13.3	9.5	9.5	19.5	19.5
Gauge Pressure (kPa)		165.432	167.273	137.512	101.946	88.329	84.059	65.396	79.052	37.127	11.680	56.653	56.653
Gauge Pressure (psi)		23.993	24.260	19.944	14.785	12.811	12.191	9.485	11.465	5.385	1.694	8.217	8.217
Total Pump Head (m)		14.39	17.79	18.58	19.19	19.23	17.81	14.08	11.21	10.76	44.18	25.89	25.89
Total Energy Out (kWh)		290.73	409.37	439.12	462.35	463.93	410.27	280.49	192.95	180.15	640.03	275.79	275.79
Efficiency		69.2%	69.2%	69.2%	69.2%	69.2%	69.2%	69.2%	69.2%	69.2%	69.2%	69.2%	69.2%
Total Energy In (kWh)		420.20	591.69	634.68	668.26	670.54	592.99	405.41	278.88	260.38	925.07	398.62	398.62
Total Paraíso Flow (m^3/s)		2.06	2.35	2.41	2.46	2.46	2.35	2.03	1.76	1.71	1.48	1.09	1.09
Total Paraíso Flow (gpm)		32,669.13	37,231.13	38,231.13	38,981.13	39,031.13	37,262.13	32,220.13	27,831.12	27,081.12	23,431.14	17,231.13	17,231.13
Total Paraíso Flow (MGD)		47,043,541	53,612,824	55,052,825	56,132,825	56,204,825	53,657,464	46,396,981	40,076,818	38,996,817	33,740,841	24,812,830	24,812,830
Gamboa Flow (gpm)		5,380	5,380	5,380	5,380	5,380	5,380	5,380	5,380	5,380	5,380	5,380	5,380
Gamboa Flow (MGD)		7,747,200	7,747,200	7,747,200	7,747,200	7,747,200	7,747,200	7,747,200	7,747,200	7,747,200	7,747,200	7,747,200	7,747,200
RW Catchment Flow (MGD)		15209280	8640000	7200000	6120000	6048000	8595360	15855840	22176000	23256000	28512000	37440000	37440000
Total Flow (MGD)		70,000,021	70,000,024	70,000,025	70,000,025	70,000,025	70,000,024	70,000,021	70,000,018	70,000,017	70,000,041	70,000,030	70,000,030
Monthy Energy (kWh)		302546.44	426014.15	456969.23	481149.06	482790.81	426951.99	291893.71	200796.27	187473.35	666053.85	287006.34	287006.34
Monthy Operating Cost		$ 42,356.50	$ 59,641.98	$ 63,975.69	$ 67,360.87	$ 67,590.71	$ 59,773.28	$ 40,865.12	$ 28,111.48	$ 26,246.27	$ 93,247.54	$ 40,180.89	$ 40,180.89
Total Annual Cost	$ 629,531												

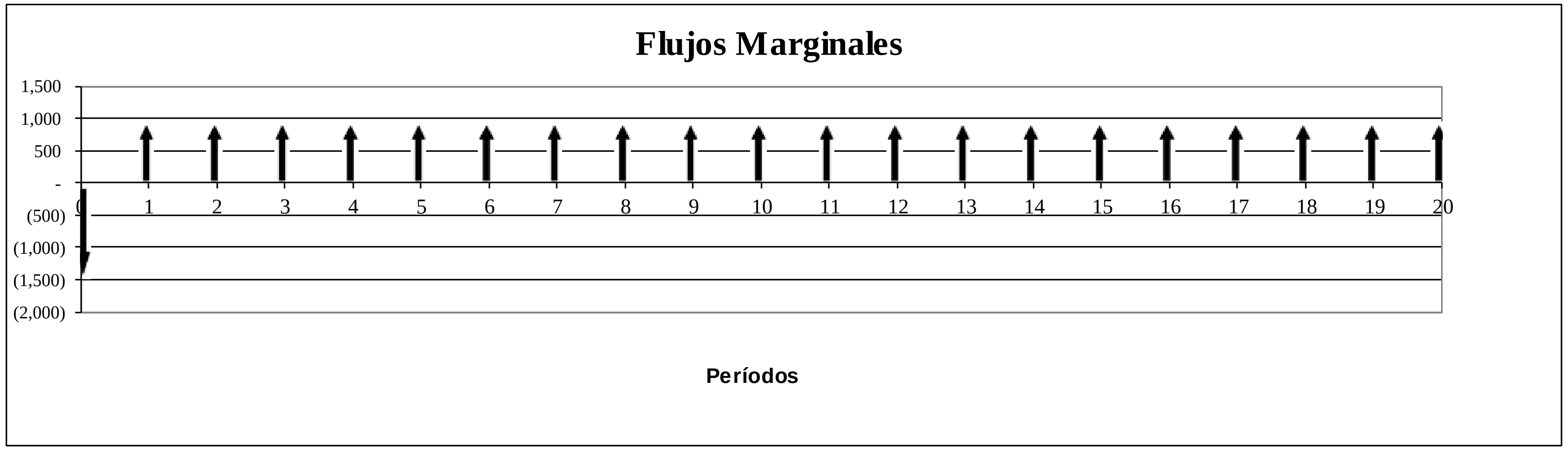
Figure 35 - Calculation Table for a 70 MGD Demand on a 36 inch 3rd Fiberglass Pipeline with a connection to the Pedro Miguel Rainwater Catchment

Appendix B – Internal Rate of Return and Actual Net Value Calculations

Nombre del proyecto: (NIP- XXXX)
(in thousands of dollars)

Ciclo presupuestario	**AF-2013**
Tasa mínima de retorno	14%

Alternativa A: 30" Third Line for 50 MGD



Método del Valor Actual Neto (VAN) y Tasa Interna de Retorno (TIR)

Method of Actual Net Value (ANV) and Internal Rate of Return (IRR)

		Alternative A							Status Quo						
Year (n)	Fiscal Year	A Investment (-)	B Expenses (-)	C Income (+)	D Loss (-)	Total Cash Flow	Present Value Factor	Present Value	A Investment (-)	B Expenses (-)	C Income (+)	D Loss (-)	Total Cash Flow	Present Value Factor	Present Value
0	2013	(1,482)	-	-	-	(1,482)	1.0000	(1,482.00)	-		-	-	-	1.0000	-
1	2014	-	(609)	-	-	(609)	0.8772	(534.21)	-	(1,562)	-	-	(1,562)	0.8772	(1,370.18)
2	2015	-	(609)	-	-	(609)	0.7695	(468.61)	-	(1,562)	-	-	(1,562)	0.7695	(1,201.91)
3	2016	-	(609)	-	-	(609)	0.6750	(411.06)	-	(1,562)	-	-	(1,562)	0.6750	(1,054.31)
4	2017	-	(609)	-	-	(609)	0.5921	(360.58)	-	(1,562)	-	-	(1,562)	0.5921	(924.83)
5	2018	-	(609)	-	-	(609)	0.5194	(316.30)	-	(1,562)	-	-	(1,562)	0.5194	(811.25)
6	2019	-	(609)	-	-	(609)	0.4556	(277.45)	-	(1,562)	-	-	(1,562)	0.4556	(711.63)
7	2020	-	(609)	-	-	(609)	0.3996	(243.38)	-	(1,562)	-	-	(1,562)	0.3996	(624.23)
8	2021	-	(609)	-	-	(609)	0.3506	(213.49)	-	(1,562)	-	-	(1,562)	0.3506	(547.57)
9	2022	-	(609)	-	-	(609)	0.3075	(187.27)	-	(1,562)	-	-	(1,562)	0.3075	(480.33)
10	2023	-	(609)	-	-	(609)	0.2697	(164.27)	-	(1,562)	-	-	(1,562)	0.2697	(421.34)
11	2024	-	(609)	-	-	(609)	0.2366	(144.10)	-	(1,562)	-	-	(1,562)	0.2366	(369.60)
12	2025	-	(609)	-	-	(609)	0.2076	(126.40)	-	(1,562)	-	-	(1,562)	0.2076	(324.21)
13	2026	-	(609)	-	-	(609)	0.1821	(110.88)	-	(1,562)	-	-	(1,562)	0.1821	(284.39)
14	2027	-	(609)	-	-	(609)	0.1597	(97.26)	-	(1,562)	-	-	(1,562)	0.1597	(249.47)
15	2028	-	(609)	-	-	(609)	0.1401	(85.32)	-	(1,562)	-	-	(1,562)	0.1401	(218.83)
16	2029	-	(609)	-	-	(609)	0.1229	(74.84)	-	(1,562)	-	-	(1,562)	0.1229	(191.96)
17	2030	-	(609)	-	-	(609)	0.1078	(65.65)	-	(1,562)	-	-	(1,562)	0.1078	(168.38)
18	2031	-	(609)	-	-	(609)	0.0946	(57.59)	-	(1,562)	-	-	(1,562)	0.0946	(147.70)
19	2032	-	(609)	-	-	(609)	0.0829	(50.52)	-	(1,562)	-	-	(1,562)	0.0829	(129.57)
20	2033	-	(609)	-	-	(609)	0.0728	(44.31)	-	(1,562)	-	-	(1,562)	0.0728	(113.65)
Totals		(1,482)	(12,180)	-	-	(13,662)		(5,515)	-	(31,240)	-	-	(31,240)		(10,345)

	Alt. A vs. Status Quo	
Year (n)	Difference in Cash Flow	Difference in Present Values
0	(1,482.00)	(1,482.00)
1	953.00	835.96
2	953.00	733.30
3	953.00	643.25
4	953.00	564.25
5	953.00	494.96
6	953.00	434.17
7	953.00	380.85
8	953.00	334.08
9	953.00	293.06
10	953.00	257.07
11	953.00	225.50
12	953.00	197.80
13	953.00	173.51
14	953.00	152.20
15	953.00	133.51
16	953.00	117.12
17	953.00	102.73
18	953.00	90.12
19	953.00	79.05
20	953.00	69.34
Totals	17,578	4,830
	64%	4,830

Figure 36 – Alternative A - 50 MGD Third 30" diameter pipeline IRR and ANV Value calculation

ACP (1716)
v.21-10-2009

AUTORIDAD DEL CANAL DE PANAMÁ

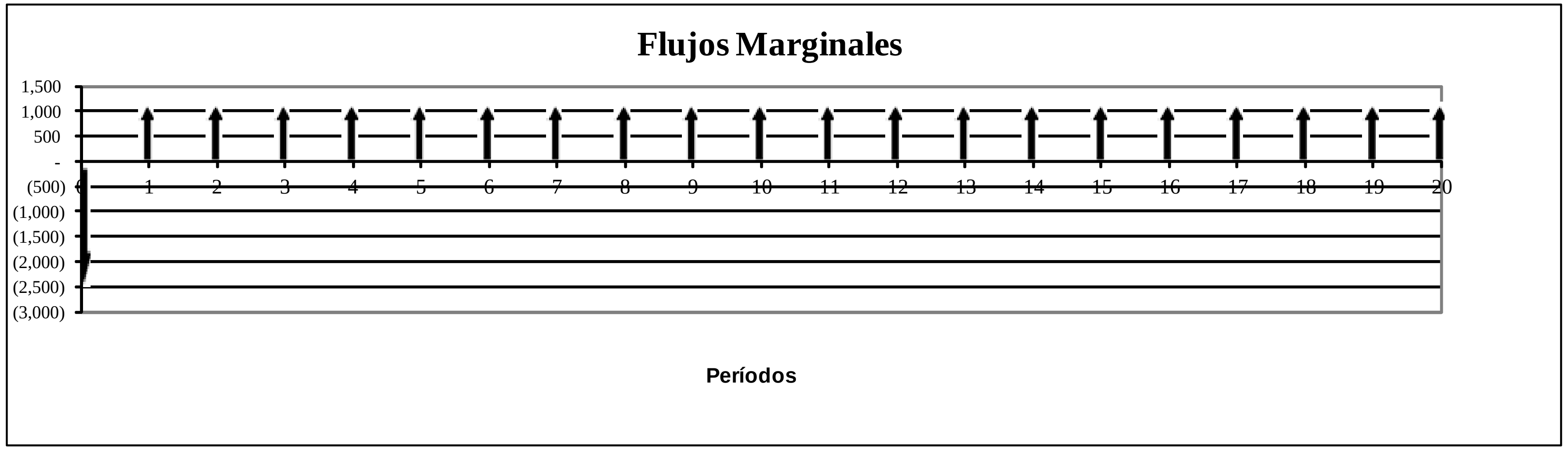
ANÁLISIS ECONÓMICO DEL FLUJO DE EFECTIVO

Nombre del proyecto: (NIP- XXXX)

(in thousands of dollars)

Ciclo presupuestario	AF-2013
Tasa mínima de retorno	14%

Alternativa B: 36" Third Line for 50 MGD



Método del Valor Actual Neto (VAN) y Tasa Interna de Retorno (TIR)

Method of Actual Net Value (ANV) and Internal Rate of Return (IRR)

		Alternative A							Status Quo						
Year (n)	Fiscal Year	A Investment (-)	B Expenses (-)	C Income (+)	D Loss (-)	Total Cash Flow	Present Value Factor	Present Value	A Investment (-)	B Expenses (-)	C Income (+)	D Loss (-)	Total Cash Flow	Present Value Factor	Present Value
0	2013	(2,519)	-	-	-	(2,519)	1.0000	(2,519.00)	-		-	-	-	1.0000	-
1	2014	-	(391)	-	-	(391)	0.8772	(342.98)	-	(1,562)	-	-	(1,562)	0.8772	(1,370.18)
2	2015	-	(391)	-	-	(391)	0.7695	(300.86)	-	(1,562)	-	-	(1,562)	0.7695	(1,201.91)
3	2016	-	(391)	-	-	(391)	0.6750	(263.91)	-	(1,562)	-	-	(1,562)	0.6750	(1,054.31)
4	2017	-	(391)	-	-	(391)	0.5921	(231.50)	-	(1,562)	-	-	(1,562)	0.5921	(924.83)
5	2018	-	(391)	-	-	(391)	0.5194	(203.07)	-	(1,562)	-	-	(1,562)	0.5194	(811.25)
6	2019	-	(391)	-	-	(391)	0.4556	(178.13)	-	(1,562)	-	-	(1,562)	0.4556	(711.63)
7	2020	-	(391)	-	-	(391)	0.3996	(156.26)	-	(1,562)	-	-	(1,562)	0.3996	(624.23)
8	2021	-	(391)	-	-	(391)	0.3506	(137.07)	-	(1,562)	-	-	(1,562)	0.3506	(547.57)
9	2022	-	(391)	-	-	(391)	0.3075	(120.24)	-	(1,562)	-	-	(1,562)	0.3075	(480.33)
10	2023	-	(391)	-	-	(391)	0.2697	(105.47)	-	(1,562)	-	-	(1,562)	0.2697	(421.34)
11	2024	-	(391)	-	-	(391)	0.2366	(92.52)	-	(1,562)	-	-	(1,562)	0.2366	(369.60)
12	2025	-	(391)	-	-	(391)	0.2076	(81.16)	-	(1,562)	-	-	(1,562)	0.2076	(324.21)
13	2026	-	(391)	-	-	(391)	0.1821	(71.19)	-	(1,562)	-	-	(1,562)	0.1821	(284.39)
14	2027	-	(391)	-	-	(391)	0.1597	(62.45)	-	(1,562)	-	-	(1,562)	0.1597	(249.47)
15	2028	-	(391)	-	-	(391)	0.1401	(54.78)	-	(1,562)	-	-	(1,562)	0.1401	(218.83)
16	2029	-	(391)	-	-	(391)	0.1229	(48.05)	-	(1,562)	-	-	(1,562)	0.1229	(191.96)
17	2030	-	(391)	-	-	(391)	0.1078	(42.15)	-	(1,562)	-	-	(1,562)	0.1078	(168.38)
18	2031	-	(391)	-	-	(391)	0.0946	(36.97)	-	(1,562)	-	-	(1,562)	0.0946	(147.70)
19	2032	-	(391)	-	-	(391)	0.0829	(32.43)	-	(1,562)	-	-	(1,562)	0.0829	(129.57)
20	2033	-	(391)	-	-	(391)	0.0728	(28.45)	-	(1,562)	-	-	(1,562)	0.0728	(113.65)
Totals		(2,519)	(7,820)	-	-	(10,339)		(5,109)	-	(31,240)	-	-	(31,240)		(10,345)

	Alt. A vs. Status Quo	
Year (n)	Difference in Cash Flow	Difference in Present Values
0	(2,519.00)	(2,519.00)
1	1,171.00	1,027.19
2	1,171.00	901.05
3	1,171.00	790.39
4	1,171.00	693.33
5	1,171.00	608.18
6	1,171.00	533.49
7	1,171.00	467.98
8	1,171.00	410.50
9	1,171.00	360.09
10	1,171.00	315.87
11	1,171.00	277.08
12	1,171.00	243.05
13	1,171.00	213.20
14	1,171.00	187.02
15	1,171.00	164.05
16	1,171.00	143.91
17	1,171.00	126.23
18	1,171.00	110.73
19	1,171.00	97.13
20	1,171.00	85.20
Totals	20,901	5,237
	46%	5,237
	IRR	ANV

Figure 37 -Alternative B - 50 MGD Third 36" diameter pipeline IRR and ANV Value calculation

AUTORIDAD DEL CANAL DE PANAMÁ

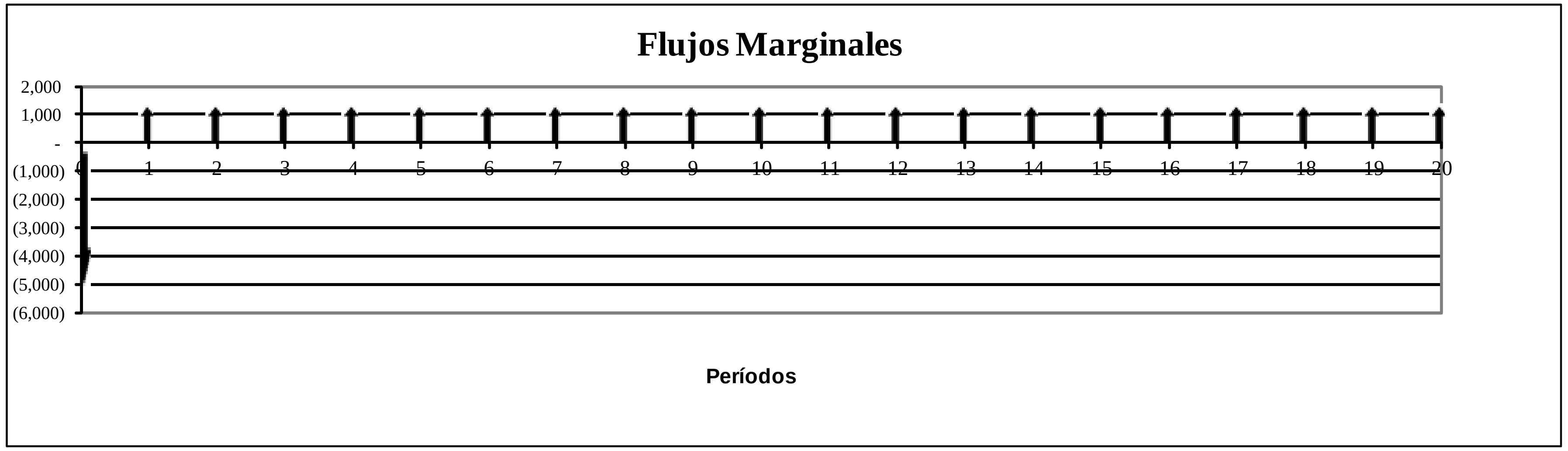
ANÁLISIS ECONÓMICO DEL FLUJO DE EFECTIVO

Nombre del proyecto: (NIP- XXXX)

(in thousands of dollars)

Ciclo presupuestario	**AF-2013**
Tasa mínima de retorno	14%

Alternative C: Third Line (30") and Rainwater Catchment for 50 MGD



Método del Valor Actual Neto (VAN) y Tasa Interna de Retorno (TIR)

Method of Actual Net Value (ANV) and Internal Rate of Return (IRR)

		Alternative A							Status Quo						
Year (n)	Fiscal Year	A Investment (-)	B Expenses (-)	C Income (+)	D Loss (-)	Total Cash Flow	Present Value Factor	Present Value	A Investment (-)	B Expenses (-)	C Income (+)	D Loss (-)	Total Cash Flow	Present Value Factor	Present Value
0	2013	(5,212)	-	-	-	(5,212)	1.0000	(5,212.00)	-		-	-	-	1.0000	-
1	2014	-	(209)	-	-	(209)	0.8772	(183.33)	-	(1,562)	-	-	(1,562)	0.8772	(1,370.18)
2	2015	-	(209)	-	-	(209)	0.7695	(160.82)	-	(1,562)	-	-	(1,562)	0.7695	(1,201.91)
3	2016	-	(209)	-	-	(209)	0.6750	(141.07)	-	(1,562)	-	-	(1,562)	0.6750	(1,054.31)
4	2017	-	(209)	-	-	(209)	0.5921	(123.74)	-	(1,562)	-	-	(1,562)	0.5921	(924.83)
5	2018	-	(209)	-	-	(209)	0.5194	(108.55)	-	(1,562)	-	-	(1,562)	0.5194	(811.25)
6	2019	-	(209)	-	-	(209)	0.4556	(95.22)	-	(1,562)	-	-	(1,562)	0.4556	(711.63)
7	2020	-	(209)	-	-	(209)	0.3996	(83.52)	-	(1,562)	-	-	(1,562)	0.3996	(624.23)
8	2021	-	(209)	-	-	(209)	0.3506	(73.27)	-	(1,562)	-	-	(1,562)	0.3506	(547.57)
9	2022	-	(209)	-	-	(209)	0.3075	(64.27)	-	(1,562)	-	-	(1,562)	0.3075	(480.33)
10	2023	-	(209)	-	-	(209)	0.2697	(56.38)	-	(1,562)	-	-	(1,562)	0.2697	(421.34)
11	2024	-	(209)	-	-	(209)	0.2366	(49.45)	-	(1,562)	-	-	(1,562)	0.2366	(369.60)
12	2025	-	(209)	-	-	(209)	0.2076	(43.38)	-	(1,562)	-	-	(1,562)	0.2076	(324.21)
13	2026	-	(209)	-	-	(209)	0.1821	(38.05)	-	(1,562)	-	-	(1,562)	0.1821	(284.39)
14	2027	-	(209)	-	-	(209)	0.1597	(33.38)	-	(1,562)	-	-	(1,562)	0.1597	(249.47)
15	2028	-	(209)	-	-	(209)	0.1401	(29.28)	-	(1,562)	-	-	(1,562)	0.1401	(218.83)
16	2029	-	(209)	-	-	(209)	0.1229	(25.68)	-	(1,562)	-	-	(1,562)	0.1229	(191.96)
17	2030	-	(209)	-	-	(209)	0.1078	(22.53)	-	(1,562)	-	-	(1,562)	0.1078	(168.38)
18	2031	-	(209)	-	-	(209)	0.0946	(19.76)	-	(1,562)	-	-	(1,562)	0.0946	(147.70)
19	2032	-	(209)	-	-	(209)	0.0829	(17.34)	-	(1,562)	-	-	(1,562)	0.0829	(129.57)
20	2033	-	(209)	-	-	(209)	0.0728	(15.21)	-	(1,562)	-	-	(1,562)	0.0728	(113.65)
Totals		(5,212)	(4,180)	-	-	(9,392)		(6,596)	-	(31,240)	-	-	(31,240)		(10,345)

	Alt. A vs. Status Quo	
Year (n)	Difference in Cash Flow	Difference in Present Values
0	(5,212.00)	(5,212.00)
1	1,353.00	1,186.84
2	1,353.00	1,041.09
3	1,353.00	913.24
4	1,353.00	801.08
5	1,353.00	702.71
6	1,353.00	616.41
7	1,353.00	540.71
8	1,353.00	474.31
9	1,353.00	416.06
10	1,353.00	364.96
11	1,353.00	320.14
12	1,353.00	280.83
13	1,353.00	246.34
14	1,353.00	216.09
15	1,353.00	189.55
16	1,353.00	166.27
17	1,353.00	145.85
18	1,353.00	127.94
19	1,353.00	112.23
20	1,353.00	98.45
Totals	21,848	3,749
	26%	3,749
	IRR	ANV

Figure 38 - Alternative C - 50 MGD Rainwater Catchment connection to a Third 30" diameter pipeline IRR and ANV Value calculation

ACP (1716)
v.21-10-2009

AUTORIDAD DEL CANAL DE PANAMÁ

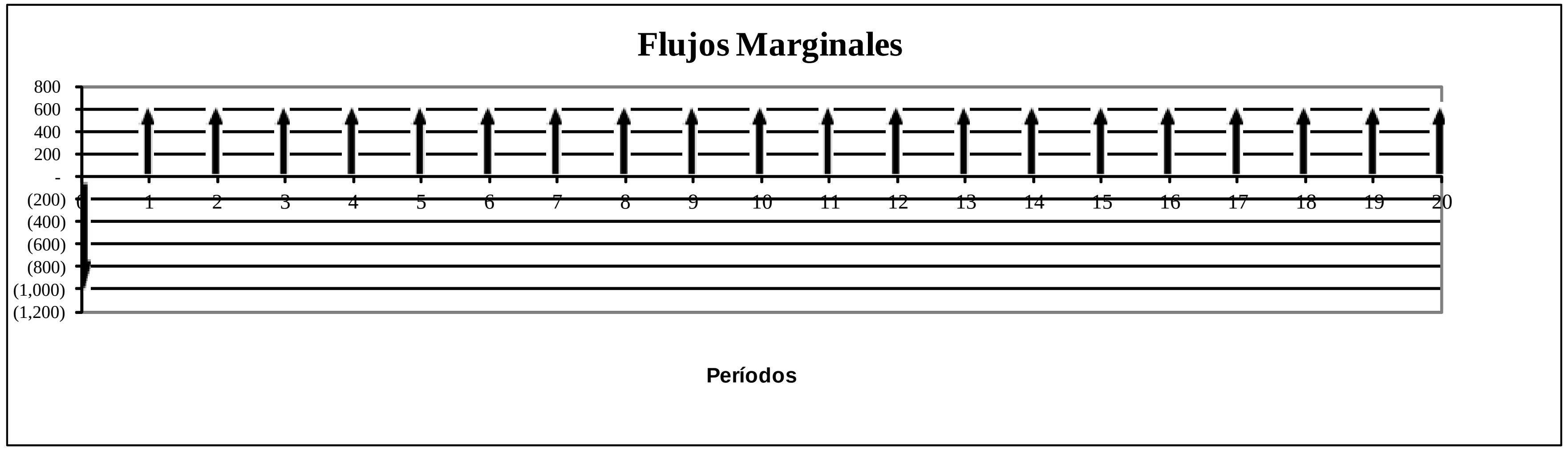
ANÁLISIS ECONÓMICO DEL FLUJO DE EFECTIVO

Nombre del proyecto: (NIP- XXXX)

(in thousands of dollars)

Ciclo presupuestario	AF-2013
Tasa mínima de retorno	14%

Alternative D: Third Line (36 in diameter) for 70 MGD



Método del Valor Actual Neto (VAN) y Tasa Interna de Retorno (TIR)

Method of Actual Net Value (ANV) and Internal Rate of Return (IRR)

		Alternative A							Status Quo						
Year (n)	Fiscal Year	A Investment (-)	B Expenses (-)	C Income (+)	D Loss (-)	Total Cash Flow	Present Value Factor	Present Value	A Investment (-)	B Expenses (-)	C Income (+)	D Loss (-)	Total Cash Flow	Present Value Factor	Present Value
0	2013	(2,519)	-	-	-	(2,519)	1.0000	(2,519.00)	(1,482)		-	-	(1,482)	1.0000	(1,482.00)
1	2014	-	(1,081)	-	-	(1,081)	0.8772	(948.25)	-	(1,737)	-	-	(1,737)	0.8772	(1,523.68)
2	2015	-	(1,081)	-	-	(1,081)	0.7695	(831.79)	-	(1,737)	-	-	(1,737)	0.7695	(1,336.57)
3	2016	-	(1,081)	-	-	(1,081)	0.6750	(729.64)	-	(1,737)	-	-	(1,737)	0.6750	(1,172.43)
4	2017	-	(1,081)	-	-	(1,081)	0.5921	(640.04)	-	(1,737)	-	-	(1,737)	0.5921	(1,028.44)
5	2018	-	(1,081)	-	-	(1,081)	0.5194	(561.44)	-	(1,737)	-	-	(1,737)	0.5194	(902.14)
6	2019	-	(1,081)	-	-	(1,081)	0.4556	(492.49)	-	(1,737)	-	-	(1,737)	0.4556	(791.35)
7	2020	-	(1,081)	-	-	(1,081)	0.3996	(432.01)	-	(1,737)	-	-	(1,737)	0.3996	(694.17)
8	2021	-	(1,081)	-	-	(1,081)	0.3506	(378.95)	-	(1,737)	-	-	(1,737)	0.3506	(608.92)
9	2022	-	(1,081)	-	-	(1,081)	0.3075	(332.42)	-	(1,737)	-	-	(1,737)	0.3075	(534.14)
10	2023	-	(1,081)	-	-	(1,081)	0.2697	(291.59)	-	(1,737)	-	-	(1,737)	0.2697	(468.54)
11	2024	-	(1,081)	-	-	(1,081)	0.2366	(255.78)	-	(1,737)	-	-	(1,737)	0.2366	(411.00)
12	2025	-	(1,081)	-	-	(1,081)	0.2076	(224.37)	-	(1,737)	-	-	(1,737)	0.2076	(360.53)
13	2026	-	(1,081)	-	-	(1,081)	0.1821	(196.82)	-	(1,737)	-	-	(1,737)	0.1821	(316.25)
14	2027	-	(1,081)	-	-	(1,081)	0.1597	(172.65)	-	(1,737)	-	-	(1,737)	0.1597	(277.42)
15	2028	-	(1,081)	-	-	(1,081)	0.1401	(151.44)	-	(1,737)	-	-	(1,737)	0.1401	(243.35)
16	2029	-	(1,081)	-	-	(1,081)	0.1229	(132.85)	-	(1,737)	-	-	(1,737)	0.1229	(213.46)
17	2030	-	(1,081)	-	-	(1,081)	0.1078	(116.53)	-	(1,737)	-	-	(1,737)	0.1078	(187.25)
18	2031	-	(1,081)	-	-	(1,081)	0.0946	(102.22)	-	(1,737)	-	-	(1,737)	0.0946	(164.25)
19	2032	-	(1,081)	-	-	(1,081)	0.0829	(89.67)	-	(1,737)	-	-	(1,737)	0.0829	(144.08)
20	2033	-	(1,081)	-	-	(1,081)	0.0728	(78.66)	-	(1,737)	-	-	(1,737)	0.0728	(126.39)
Totals		(2,519)	(21,620)	-	-	(24,139)		(9,679)	(1,482)	(34,740)	-	-	(36,222)		(12,986)

	Alt. A vs. Status Quo	
Year (n)	Difference in Cash Flow	Difference in Present Values
0	(1,037.00)	(1,037.00)
1	656.00	575.44
2	656.00	504.77
3	656.00	442.78
4	656.00	388.40
5	656.00	340.71
6	656.00	298.86
7	656.00	262.16
8	656.00	229.97
9	656.00	201.73
10	656.00	176.95
11	656.00	155.22
12	656.00	136.16
13	656.00	119.44
14	656.00	104.77
15	656.00	91.90
16	656.00	80.62
17	656.00	70.72
18	656.00	62.03
19	656.00	54.41
20	656.00	47.73
Totals	12,083	3,308
	63%	3,308
	IRR	ANV

Figure 39 - Alternative D - 70 MGD Third 36" diameter pipeline IRR and ANV Value calculation

ACP (1716)
v. 21-10-2009

AUTORIDAD DEL CANAL DE PANAMÁ

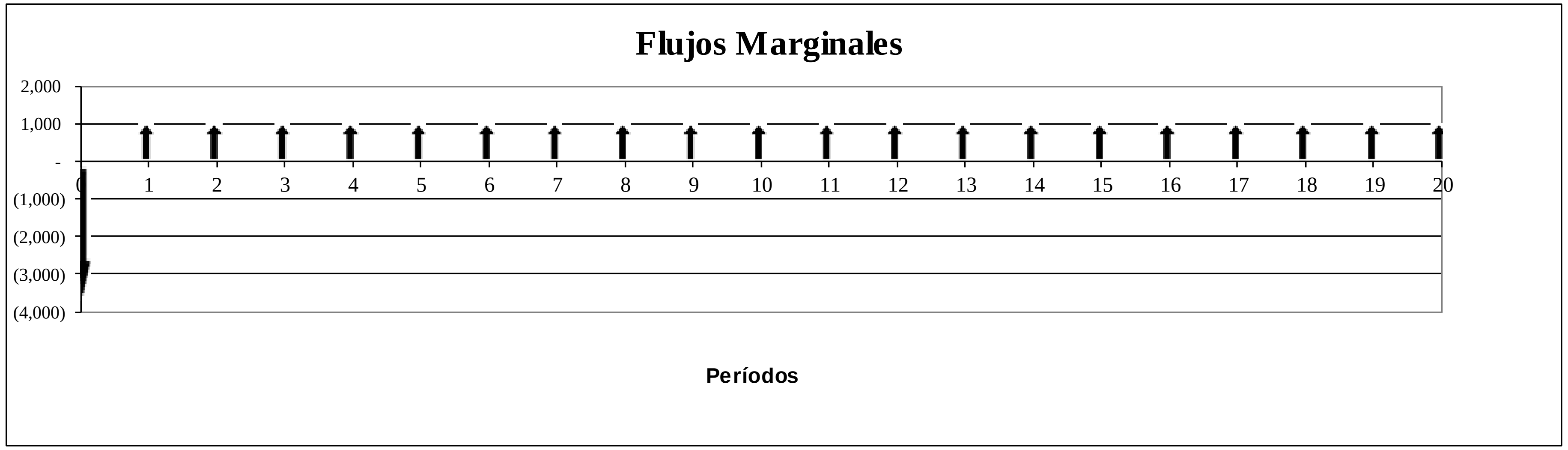
ANÁLISIS ECONÓMICO DEL FLUJO DE EFECTIVO

Nombre del proyecto: (NIP- XXXX)

(in thousands of dollars)

Ciclo presupuestario	AF-2013
Tasa mínima de retorno	14%

Alternative E: Third Line (30 in diameter) and Rainwater Catchment for 70 MGD



Método del Valor Actual Neto (VAN) y Tasa Interna de Retorno (TIR)

Method of Actual Net Value (ANV) and Internal Rate of Return (IRR)

		Alternative A							Status Quo						
Year (n)	Fiscal Year	A Investment (-)	B Expenses (-)	C Income (+)	D Loss (-)	Total Cash Flow	Present Value Factor	Present Value	A Investment (-)	B Expenses (-)	C Income (+)	D Loss (-)	Total Cash Flow	Present Value Factor	Present Value
0	2013	(5,212)	-	-	-	(5,212)	1.0000	(5,212.00)	(1,482)		-	-	(1,482)	1.0000	(1,482.00)
1	2014	-	(969)	-	-	(969)	0.8772	(850.00)	-	(1,974)	-	-	(1,974)	0.8772	(1,731.58)
2	2015	-	(969)	-	-	(969)	0.7695	(745.61)	-	(1,974)	-	-	(1,974)	0.7695	(1,518.93)
3	2016	-	(969)	-	-	(969)	0.6750	(654.05)	-	(1,974)	-	-	(1,974)	0.6750	(1,332.39)
4	2017	-	(969)	-	-	(969)	0.5921	(573.73)	-	(1,974)	-	-	(1,974)	0.5921	(1,168.77)
5	2018	-	(969)	-	-	(969)	0.5194	(503.27)	-	(1,974)	-	-	(1,974)	0.5194	(1,025.23)
6	2019	-	(969)	-	-	(969)	0.4556	(441.46)	-	(1,974)	-	-	(1,974)	0.4556	(899.33)
7	2020	-	(969)	-	-	(969)	0.3996	(387.25)	-	(1,974)	-	-	(1,974)	0.3996	(788.88)
8	2021	-	(969)	-	-	(969)	0.3506	(339.69)	-	(1,974)	-	-	(1,974)	0.3506	(692.00)
9	2022	-	(969)	-	-	(969)	0.3075	(297.98)	-	(1,974)	-	-	(1,974)	0.3075	(607.02)
10	2023	-	(969)	-	-	(969)	0.2697	(261.38)	-	(1,974)	-	-	(1,974)	0.2697	(532.47)
11	2024	-	(969)	-	-	(969)	0.2366	(229.28)	-	(1,974)	-	-	(1,974)	0.2366	(467.08)
12	2025	-	(969)	-	-	(969)	0.2076	(201.12)	-	(1,974)	-	-	(1,974)	0.2076	(409.72)
13	2026	-	(969)	-	-	(969)	0.1821	(176.43)	-	(1,974)	-	-	(1,974)	0.1821	(359.40)
14	2027	-	(969)	-	-	(969)	0.1597	(154.76)	-	(1,974)	-	-	(1,974)	0.1597	(315.27)
15	2028	-	(969)	-	-	(969)	0.1401	(135.75)	-	(1,974)	-	-	(1,974)	0.1401	(276.55)
16	2029	-	(969)	-	-	(969)	0.1229	(119.08)	-	(1,974)	-	-	(1,974)	0.1229	(242.59)
17	2030	-	(969)	-	-	(969)	0.1078	(104.46)	-	(1,974)	-	-	(1,974)	0.1078	(212.80)
18	2031	-	(969)	-	-	(969)	0.0946	(91.63)	-	(1,974)	-	-	(1,974)	0.0946	(186.66)
19	2032	-	(969)	-	-	(969)	0.0829	(80.38)	-	(1,974)	-	-	(1,974)	0.0829	(163.74)
20	2033	-	(969)	-	-	(969)	0.0728	(70.51)	-	(1,974)	-	-	(1,974)	0.0728	(143.63)
	Totals	(5,212)	(19,380)	-	-	(24,592)		(11,630)	(1,482)	(39,480)	-	-	(40,962)		(14,556)

	Alt. A vs. Status Quo	
Year (n)	Difference in Cash Flow	Difference in Present Values
0	(3,730.00)	(3,730.00)
1	1,005.00	881.58
2	1,005.00	773.31
3	1,005.00	678.35
4	1,005.00	595.04
5	1,005.00	521.97
6	1,005.00	457.86
7	1,005.00	401.64
8	1,005.00	352.31
9	1,005.00	309.05
10	1,005.00	271.09
11	1,005.00	237.80
12	1,005.00	208.60
13	1,005.00	182.98
14	1,005.00	160.51
15	1,005.00	140.80
16	1,005.00	123.51
17	1,005.00	108.34
18	1,005.00	95.03
19	1,005.00	83.36
20	1,005.00	73.13
Totals	16,370	2,926
	27%	2,926
	IRR	ANV

Figure 40 - Alternative E - 70 MGD Rainwater Catchment connection to a Third 30" diameter pipeline IRR and ANV Value calculation

ACP (1716)
v.21-10-2009

AUTORIDAD DEL CANAL DE PANAMÁ

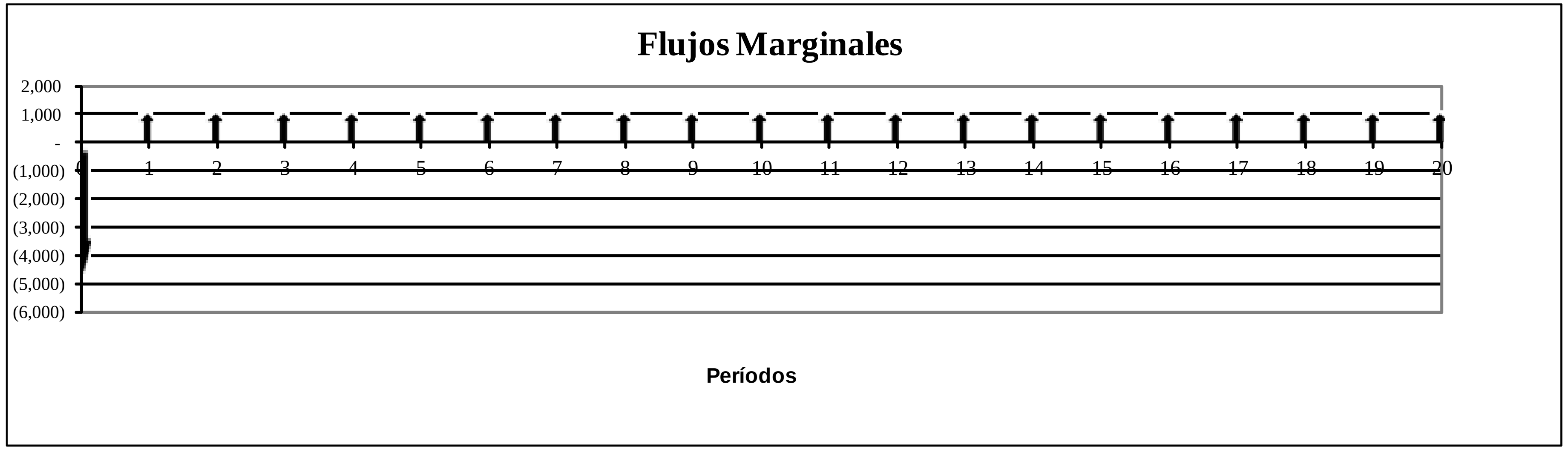
ANÁLISIS ECONÓMICO DEL FLUJO DE EFECTIVO

Nombre del proyecto: (NIP- XXXX)

(in thousands of balboas)

Ciclo presupuestario	AF-2013
Tasa mínima de retorno	14%

Alternativa F: Third Line (36 in diameter) and Rainwater Catchment for 70 MGD



Método del Valor Actual Neto (VAN) y Tasa Interna de Retorno (TIR)

Method of Actual Net Value (ANV) and Internal Rate of Return (IRR)

		Alternative A							Status Quo						
Year (n)	Fiscal Year	A Investment (-)	B Expenses (-)	C Income (+)	D Loss (-)	Total Cash Flow	Present Value Factor	Present Value	A Investment (-)	B Expenses (-)	C Income (+)	D Loss (-)	Total Cash Flow	Present Value Factor	Present Value
0	2013	(6,249)	-	-	-	(6,249)	1.0000	(6,249.00)	(1,482)		-	-	(1,482)	1.0000	(1,482.00)
1	2014	-	(630)	-	-	(630)	0.8772	(552.63)	-	(1,737)	-	-	(1,737)	0.8772	(1,523.68)
2	2015	-	(630)	-	-	(630)	0.7695	(484.76)	-	(1,737)	-	-	(1,737)	0.7695	(1,336.57)
3	2016	-	(630)	-	-	(630)	0.6750	(425.23)	-	(1,737)	-	-	(1,737)	0.6750	(1,172.43)
4	2017	-	(630)	-	-	(630)	0.5921	(373.01)	-	(1,737)	-	-	(1,737)	0.5921	(1,028.44)
5	2018	-	(630)	-	-	(630)	0.5194	(327.20)	-	(1,737)	-	-	(1,737)	0.5194	(902.14)
6	2019	-	(630)	-	-	(630)	0.4556	(287.02)	-	(1,737)	-	-	(1,737)	0.4556	(791.35)
7	2020	-	(630)	-	-	(630)	0.3996	(251.77)	-	(1,737)	-	-	(1,737)	0.3996	(694.17)
8	2021	-	(630)	-	-	(630)	0.3506	(220.85)	-	(1,737)	-	-	(1,737)	0.3506	(608.92)
9	2022	-	(630)	-	-	(630)	0.3075	(193.73)	-	(1,737)	-	-	(1,737)	0.3075	(534.14)
10	2023	-	(630)	-	-	(630)	0.2697	(169.94)	-	(1,737)	-	-	(1,737)	0.2697	(468.54)
11	2024	-	(630)	-	-	(630)	0.2366	(149.07)	-	(1,737)	-	-	(1,737)	0.2366	(411.00)
12	2025	-	(630)	-	-	(630)	0.2076	(130.76)	-	(1,737)	-	-	(1,737)	0.2076	(360.53)
13	2026	-	(630)	-	-	(630)	0.1821	(114.70)	-	(1,737)	-	-	(1,737)	0.1821	(316.25)
14	2027	-	(630)	-	-	(630)	0.1597	(100.62)	-	(1,737)	-	-	(1,737)	0.1597	(277.42)
15	2028	-	(630)	-	-	(630)	0.1401	(88.26)	-	(1,737)	-	-	(1,737)	0.1401	(243.35)
16	2029	-	(630)	-	-	(630)	0.1229	(77.42)	-	(1,737)	-	-	(1,737)	0.1229	(213.46)
17	2030	-	(630)	-	-	(630)	0.1078	(67.91)	-	(1,737)	-	-	(1,737)	0.1078	(187.25)
18	2031	-	(630)	-	-	(630)	0.0946	(59.57)	-	(1,737)	-	-	(1,737)	0.0946	(164.25)
19	2032	-	(630)	-	-	(630)	0.0829	(52.26)	-	(1,737)	-	-	(1,737)	0.0829	(144.08)
20	2033	-	(630)	-	-	(630)	0.0728	(45.84)	-	(1,737)	-	-	(1,737)	0.0728	(126.39)
Totals		(6,249)	(12,600)	-	-	(18,849)		(10,422)	(1,482)	(34,740)	-	-	(36,222)		(12,986)

	Alt. A vs. Status Quo	
Year (n)	Difference in Cash Flow	Difference in Present Values
0	(4,767.00)	(4,767.00)
1	1,107.00	971.05
2	1,107.00	851.80
3	1,107.00	747.19
4	1,107.00	655.43
5	1,107.00	574.94
6	1,107.00	504.33
7	1,107.00	442.40
8	1,107.00	388.07
9	1,107.00	340.41
10	1,107.00	298.61
11	1,107.00	261.94
12	1,107.00	229.77
13	1,107.00	201.55
14	1,107.00	176.80
15	1,107.00	155.09
16	1,107.00	136.04
17	1,107.00	119.33
18	1,107.00	104.68
19	1,107.00	91.82
20	1,107.00	80.55
Totals	17,373	2,565
	23%	2,565
	IRR	ANV

Figure 41 - Alternative F - 70 MGD Rainwater Catchment connection to a Third 36" diameter pipeline IRR and ANV Value calculation

Appendix C – Third Pipeline Design Calculations

Parameters	Symbol	Unitless	English Units				Metric Units	
Pipe Diameter	D		30	inches	2.5	feet	0.762	meters
Pipe Area	A		706.8583471	in^2	4.908738521	ft^2	0.45604	m^2
Absolute Pipe Roughness (Old Pipe)	ε_{old}		0.01	inches	0.000833333	feet	0.00025	meters
Absolute Pipe Roughness (Fiberglass)	$\varepsilon_{fiberglass}$		0.00021	inches	0.0000175	feet	5.3E-06	meters
Pipe Length (North Line)	L_{north}				14878.6094	feet	4535	meters
Pipe Length (South Line)	L_{south}				15583.99	feet	4750	meters
Fluid Density	ρ				62.4	lb/ft^3	999.24	kg/m^3
Fluid Viscosity	μ				15.04577942	ft^2/sec	1.3978	m^2/sec
Specific Gravity	SG	1						
Gravitation Constant	g				32.2	ft/sec^2	9.81	m/sec^2
Specific Weight	γ				62.42	lb/ft^3	9.806	kN/m^3
Hazen-Williams Roughness Coefficient (Old Pipe)	C_{old}	75						
Hazen-Williams Roughness Coefficient (Fiberglass)	$C_{fiberglass}$	150						

Figure 42 - Known Information for Calculations of Third Pipeline

	Alternative	North Line Flow (gpm)	North Line Flow (m^3/sec)	South Line Flow (gpm)	South Line Flow (m^3/sec)	Third Line Flow (gpm)	Third Line Flow (m^3/sec)
50 MGD Demand							
	Current	13040	0.82	13252	0.84	-	-
A	30" Diameter 3rd Line	7065	0.45	7180	0.45	15120	0.95
B	36" Diameter 3rd Line	5360	0.34	5447	0.34	18536	1.17
C	Rainwater Catchment to 30" 3rd Lin	1658 to 6050	0.10 to 0.38	1685 to 6148	0.11 to 0.39	6790 to 12945	0.43 to 0.82
	January	4518	0.285	4591	0.29	9670	0.61
	February	5617	0.354	5708	0.36	12020	0.758
	March	5858	0.37	5953	0.376	12535	0.791
	April	6038	0.381	6136	0.387	12920	0.815
	May	6050	0.382	6148	0.388	12945	0.817
	June	5625	0.355	5716	0.361	12033	0.759
	July	4410	0.278	4482	0.283	9440	0.596
	August	3355	0.212	3410	0.215	7180	0.453
	September	3175	0.2	3227	0.204	6790	0.428
	October	4733	0.299	4810	0.303	0	0
	November	1658	0.105	1685	0.106	0	0
	December	1658	0.105	1685	0.106	0	0
70 MGD Demand							
	30" Diameter 3rd Line	10285	0.65	10452	0.66	22000	1.39
D	36" Diameter 3rd Line	7809	0.49	7936	0.50	27000.00	1.70
E	Rainwater Catchment to 30" 3rd Lin	4147 to 9392	0.26 to 0.59	4214 to 9545	0.27 to 0.60	8870 to 20095	0.56 to 1.27
	January	7860	0.496	7988	0.504	16820	1.061
	February	8960	0.565	9106	0.574	19170	1.209
	March	9200	0.58	9350	0.59	19685	1.242
	April	9380	0.592	9532	0.601	20070	1.266
	May	9392	0.592	9545	0.602	20095	1.268
	June	8967	0.566	9113	0.575	19183	1.21
	July	7752	0.489	7878	0.497	16590	1.048
	August	6695	0.422	6804	0.429	14330	0.904
	September	6516	0.411	6622	0.418	13945	0.88
	October	5638	0.356	5730	0.361	12065	0.761
	November	8546	0.539	8685	0.548	0	0
	December	8546	0.539	8685	0.548	0	0
F	Rainwater Catchment to 36" 3rd Lin	3150 to 11621	0.20 to 0.73	3201 to 11810	0.20 to 0.75	10880 to 24650	0.69 to 1.56
	January	5967	0.376	6064	0.383	20640	1.302
	February	6802	0.429	6913	0.436	23515	1.483
	March	6985	0.441	7099	0.448	24150	1.523
	April	7123	0.449	7239	0.457	24620	1.553
	May	7130	0.45	7246	0.457	24650	1.555
	June	6808	0.429	6919	0.436	23535	1.485
	July	5885	0.371	5981	0.377	20350	1.284
	August	5085	0.321	5168	0.326	17580	1.109
	September	4950	0.312	5030	0.317	17101	1.079
	October	4281	0.27	4351	0.274	14800	0.934
	November	8546	0.539	8685	0.548	0	0
	December	8546	0.539	8685	0.548	0	0

Figure 43 - Calculated Flow Values from Model Used in Third Pipeline Design

	Alternative	North Line Velocity (ft/sec)	North Line Velocity (m/sec)	South Line Velocity (ft/sec)	South Line Velocity (m/sec)	Third Line Velocity (ft/sec)	Third Line Velocity (m/sec)
50 MGD Demand							
	Current	5.919	1.798	6.015	1.842	-	-
A	30" Diameter 3rd Line	3.207	0.987	3.259	0.987	6.863	2.083
B	36" Diameter 3rd Line	1.690	0.746	1.717	0.746	5.843	2.566
C	Rainwater Catchment to 30" 3rd Lin	0.753 to 2.746	0.219 to 0.833	0.765 to 2.791	0.241 to 0.855	3.082 to 5.876	0.943 to 1.798
	January	1.424	0.625	1.447	0.636	3.048	1.338
	February	1.771	0.776	1.799	0.789	3.789	1.662
	March	1.847	0.811	1.877	0.824	3.951	1.735
	April	1.903	0.835	1.934	0.849	4.073	1.787
	May	1.907	0.838	1.938	0.851	4.081	1.792
	June	1.773	0.778	1.802	0.792	3.793	1.664
	July	1.390	0.610	1.413	0.621	2.976	1.307
	August	1.058	0.465	1.075	0.471	2.263	0.993
	September	1.001	0.439	1.017	0.447	2.140	0.939
	October	1.492	0.656	1.516	0.664	0.000	0.000
	November	0.523	0.230	0.531	0.232	0.000	0.000
	December	0.523	0.230	0.531	0.232	0.000	0.000
70 MGD Demand							
	30" Diameter 3rd Line	4.669	1.425	4.744	1.447	9.986	3.048
D	36" Diameter 3rd Line	2.462	1.074	2.502	1.096	8.511	3.728
E	Rainwater Catchment to 30" 3rd Lin	1.882 to 4.263	0.570 to 1.294	1.913 to 4.333	0.592 to 1.316	4.026 to 9.121	1.228 to 2.785
	January	2.478	1.088	2.518	1.105	5.302	2.327
	February	2.824	1.239	2.870	1.259	6.043	2.651
	March	2.900	1.272	2.947	1.294	6.205	2.723
	April	2.957	1.298	3.005	1.318	6.326	2.776
	May	2.961	1.298	3.009	1.320	6.334	2.780
	June	2.827	1.241	2.873	1.261	6.047	2.653
	July	2.444	1.072	2.483	1.090	5.230	2.298
	August	2.110	0.925	2.145	0.941	4.517	1.982
	September	2.054	0.901	2.087	0.917	4.396	1.930
	October	1.777	0.781	1.806	0.792	3.803	1.669
	November	2.694	1.182	2.738	1.202	0.000	0.000
	December	2.694	1.182	2.738	1.202	0.000	0.000
F	Rainwater Catchment to 36" 3rd Lin	1.430 to 5.275	0.439 to 1.601	1.453 to 5.361	0.439 to 1.645	4.939 to 11.189	1.513 to 3.421
	January	1.881	0.824	1.911	0.840	6.506	2.855
	February	2.144	0.941	2.179	0.956	7.412	3.252
	March	2.202	0.967	2.238	0.982	7.613	3.340
	April	2.245	0.985	2.282	1.002	7.761	3.405
	May	2.248	0.987	2.284	1.002	7.770	3.410
	June	2.146	0.941	2.181	0.956	7.419	3.256
	July	1.855	0.814	1.885	0.827	6.415	2.816
	August	1.603	0.704	1.629	0.715	5.542	2.432
	September	1.560	0.684	1.586	0.695	5.391	2.366
	October	1.349	0.592	1.372	0.601	4.665	2.048
	November	2.694	1.182	2.738	1.202	0.000	0.000
	December	2.694	1.182	2.738	1.202	0.000	0.000

Figure 44 - Velocities Calculated from the Previous Flows for Use in the Reynolds Number Equation in the Darcy-Weisbach Equation

$$d = \frac{0.73\sqrt{\frac{Q}{SG}}}{\rho^{0.33}}$$

Specific Gravity	1.0
Density	62.4 lb/ft^3

Minimum Required Diameter

		North Line		South Line		Third Line	
	Alternative	Flow (gpm)	Minimum Diameter (inches)	Flow (gpm)	Minimum Diameter (inches)	Flow (gpm)	Minimum Diameter (inches)
50 MGD Demand							
	Current	13040	32.600	13252	32.900	-	-
A	30" Diameter 3rd Line	7065	24.000	7180	24.200	15120	22.945
B	36" Diameter 3rd Line	5360	20.900	5447	21.100	18536	25.405
C	Rainwater Catchment to 30" 3rd Line	1658 to 6050	11.6 to 22.2	1685 to 6148	11.7 to 22.4	6790 to 12945	15.376 to 21.231
	January	4518	19.200	4591	19.400	9670	18.350
	February	5617	21.400	5708	21.600	12020	20.458
	March	5858	21.900	5953	22.100	12535	20.892
	April	6038	22.200	6136	22.400	12920	21.210
	May	6050	22.200	6148	22.400	12945	21.231
	June	5625	21.400	5716	21.600	12033	20.469
	July	4410	19.000	4482	19.100	9440	18.130
	August	3355	16.600	3410	16.700	7180	15.812
	September	3175	16.100	3227	16.200	6790	15.376
	October	4733	19.700	4810	19.800	0	0.000
	November	1658	11.600	1685	11.700	0	0.000
	December	1658	11.600	1685	11.700	0	0.000
70 MGD Demand							
	30" Diameter 3rd Line	10285	29.000	10452	29.200	22000	27.677
D	36" Diameter 3rd Line	7809	25.300	7936	25.500	27000.00	30.662
E	Rainwater Catchment to 30" 3rd Line	4147 to 9392	21.5 to 27.7	4214 to 9545	21.6 to 27.9	8870 to 20095	20.496 to 26.452
	January	7860	25.300	7988	25.500	16820	24.201
	February	8960	27.100	9106	27.300	19170	25.836
	March	9200	27.400	9350	27.600	19685	26.181
	April	9380	27.700	9532	27.900	20070	26.435
	May	9392	27.700	9545	27.900	20095	26.452
	June	8967	27.100	9113	27.300	19183	25.845
	July	7752	25.200	7878	25.400	16590	24.035
	August	6695	23.400	6804	23.600	14330	22.338
	September	6516	23.100	6622	23.300	13945	22.035
	October	5638	21.500	5730	21.600	12065	20.496
	November	8546	26.400	8685	26.600	0	0.000
	December	8546	26.400	8685	26.600	0	0.000
F	Rainwater Catchment to 36" 3rd Line	3150 to 11621	18.7 to 24.1	3201 to 11810	18.9 to 26.6	10880 to 24650	22.701 to 29.297
	January	5967	22.100	6064	22.300	20640	26.808
	February	6802	23.600	6913	23.800	23515	28.614
	March	6985	23.900	7099	24.100	24150	28.998
	April	7123	24.100	7239	24.300	24620	29.279
	May	7130	24.100	7246	24.300	24650	29.297
	June	6808	23.600	6919	23.800	23535	28.627
	July	5885	21.900	5981	22.100	20350	26.619
	August	5085	20.400	5168	20.500	17580	24.741
	September	4950	20.100	5030	20.300	17101	24.402
	October	4281	18.700	4351	18.900	14800	22.701
	November	8546	26.400	8685	26.600	0	0.000
	December	8546	26.400	8685	26.600	0	0.000

Figure 45 - Calculated Minimum Diameters for Each Pipeline for Different Flow Conditions

$$Nr = \frac{D * v}{\vartheta}$$

Nr =	Reynolds Number		
D =	Diameter, m		
v =	Velocity, m/sec		
ϑ =	Fluid Kinematic Viscosity, m^2/sec		1.40E-06
D =	30 inches	m	0.762
D =	36 inches	m	0.9144

Reynolds Number		North Line		South Line		Third Line	
	Alternative	Velocity (m/sec)	Reynolds Number	Velocity (m/sec)	Reynolds Number	Velocity (m/sec)	Reynolds Number
50 MGD Demand							
	Current	1.798	980166	1.842	1004152	-	-
A	30" Diameter 3rd Line	0.987	538056	0.987	538056	2.083	1135532
B	36" Diameter 3rd Line	0.746	406676	0.746	406676	1.782	1165732
C	Rainwater Catchment to 30" 3rd Line	0.219	57150	0.241	57694	0.943	224790
		0.833	213081	0.855	219891	1.798	427264
70 MGD Demand							
	30" Diameter 3rd Line	1.425	776828	1.447	788821	3.048	1661594
D	36" Diameter 3rd Line	1.074	585483	1.096	597476	2.589	1693648
E	Rainwater Catchment to 30" 3rd Line	0.57	201386	0.592	204651	1.228	398417
		1.294	335824	1.316	341267	2.785	663484
F	Rainwater Catchment to 36" 3rd Line	0.439	185493	0.439	188105	1.051	591747
		1.601	352044	1.645	357922	2.376	986246

Figure 46 - Calculated Reynolds Numbers for Each Alternative

$$\frac{1}{\sqrt{f}} = -2 * \log\left(\left(\frac{e}{3.7D}\right) + \left(\frac{2.51}{Nr * \sqrt{f}}\right)\right)$$

$$f = \frac{0.25}{\left(\log_{10}\left(\frac{\varepsilon}{3.7D} + \frac{5.74}{Re^{0.9}}\right)\right)^2}$$

The equation on the left is the Colebrook Equation. The equation on the right is the Swamee-Jain Equation. The Swamee-Jain Equation solves directly for the friction factor and is an approximation of the implicit Colebrook-White Equation.

ε	fiberglass	5.00E-06	meters
ε	cast iron	1.22E-04	meters

Friction Factor		North Line				South Line				Third Line			
	Alternative	Pipe Roughness	Pipe Diameter	Reynolds Number	Friction Factor	Pipe Roughness	Pipe Diameter	Reynolds Number	Friction Factor	Pipe Roughness	Pipe Diameter	Reynolds Number	Friction Factor
50 MGD Demand													
	Current	1.22E-04	0.762	980166	0.014329	1.22E-04	0.762	1004152	0.014307	-	-	-	-
A	30" Diameter 3rd Line	1.22E-04	0.762	538056	0.015019	1.22E-04	0.762	538056	0.015019	5.00E-06	0.9144	1135532	0.011511
B	36" Diameter 3rd Line	1.22E-04	0.762	406676	0.015439	1.22E-04	0.762	406676	0.015439	5.00E-06	0.9144	1165732	0.011464
C	Rainwater Catchment to 30" 3rd Line	1.22E-04	0.762	57150	0.020835	1.22E-04	0.762	57694	0.015439	5.00E-06	0.9144	224790	0.011464
		1.22E-04	0.762	213081	0.016699	1.22E-04	0.762	219891	0.016628	5.00E-06	0.9144	427264	0.013547
70 MGD Demand													
	30" Diameter 3rd Line	1.22E-04	0.762	776828	0.014565	1.22E-04	0.762	788821	0.014549	5.00E-06	0.9144	1661594	0.010863
D	36" Diameter 3rd Line	1.22E-04	0.762	585493	0.014905	1.22E-04	0.762	597476	0.014879	5.00E-06	0.9144	1693648	0.010832
E	Rainwater Catchment to 30" 3rd Line	1.22E-04	0.762	201386	0.016831	1.22E-04	0.762	204651	0.016793	5.00E-06	0.9144	398417	0.013716
		1.22E-04	0.762	335824	0.014905	1.22E-04	0.762	341267	0.014879	5.00E-06	0.9144	663484	0.012558
F	Rainwater Catchment to 36" 3rd Line	1.22E-04	0.762	185493	0.017029	1.22E-04	0.762	188105	0.016994	5.00E-06	0.9144	591747	0.010832
		1.22E-04	0.762	352044	0.014905	1.22E-04	0.762	357922	0.014879	5.00E-06	0.9144	986246	0.011770

Figure 47 - Calculating the Friction Factors for Each Proposed Alternative

$$H_f = f\left(\frac{L}{D}\right)\left(\frac{v^2}{2g}\right)$$

North Pipe Length	4535	m
South Pipe Length	4750	m
Third Pipe Length	4320	m
30 inch pipe diameter	0.762	m
36 inch pipe diameter	0.9144	m
Gravitational Constant	9.81	m/s^2

Major Head Loss		North Line			South Line			Third Line		
	Alternative	Friction Factor	Velocity (m/s)	Head Loss (m)	Friction Factor	Velocity (m/s)	Head Loss (m)	Friction Factor	Velocity (m/s)	Head Loss (m)
50 MGD Demand										
	Current	0.014329	1.798	14.05137	0.014307	1.842	15.42295	-	-	-
A	30" Diameter 3rd Line	0.015019	0.987	4.43812	0.015019	0.987	4.64852	0.011511	2.083	14.43183
B	36" Diameter 3rd Line	0.015439	0.746	2.60627	0.015439	0.746	2.98311	0.011464	1.782	8.76597
C	Rainwater Catchment to 30" 3rd Line	0.020835	0.219	2.84808	0.015439	0.241	0.28490	0.011464	0.943	8.76565
		0.016699	0.833	3.51483	0.016628	0.855	2.98311	0.013547	1.798	10.54559
70 MGD Demand										
	30" Diameter 3rd Line	0.014565	1.425	8.97147	0.014549	1.447	9.67854	0.010863	3.048	29.16147
D	36" Diameter 3rd Line	0.014905	1.074	5.76309	0.014879	1.096	5.67851	0.010832	2.589	17.48354
E	Rainwater Catchment to 30" 3rd Line	0.016831	0.57	1.65876	0.016793	0.592	6.27875	0.013716	1.228	4.98050
		0.014905	1.294	5.76309	0.014879	1.316	8.18701	0.012558	2.785	17.48354
F	Rainwater Catchment to 36" 3rd Line	0.017029	0.439	0.99550	0.016994	0.439	6.27875	0.010832	1.051	2.88113
		0.014905	1.601	5.76309	0.014879	1.645	12.79220	0.011770	2.376	17.48354

Figure 48 - Calculated Major Head Loss for Each Alternative Using the Darcy-Weisbach Equation

Pipe fittings such as bends cause pressure loss or resistance in a pipe network. To calculate the minor losses within the raw water system, the following equation will be used:

$$H_f = \frac{kv^2}{2g}$$

Parameters

90° Elbow	0.36	
45° Elbow	0.19	
Gravity	9.81	ft/sec^2

Minor Head Loss		North Line				South Line				Third Line			
	Alternative	90° Elbow	45° Elbow	Velocity (m/s)	Minor Losses	90° Elbow	45° Elbow	Velocity (m/s)	Minor Losses	90° Elbow	45° Elbow	Velocity (m/s)	Minor Losses
50 MGD Demand													
	Current	5	11	1.798	0.641	3	22	1.842	0.910	2	7	-	-
A	30" Diameter 3rd Line	5	11	0.987	0.193	3	22	0.987	0.261	2	7	2.083	0.453
B	36" Diameter 3rd Line	5	11	0.746	0.110	3	22	0.746	0.149	2	7	1.782	0.332
C	Rainwater Catchment to 30" 3rd Line	5	11	0.219	0.641	3	22	0.241	0.016	2	7	0.943	0.093
		5	11	0.833	0.138	3	22	0.855	0.910	2	7	1.798	0.453
70 MGD Demand													
	30" Diameter 3rd Line	5	11	1.425	0.403	3	22	1.447	0.561	2	7	3.048	0.971
D	36" Diameter 3rd Line	5	11	1.074	0.641	3	22	1.096	0.910	2	7	2.589	0.700
E	Rainwater Catchment to 30" 3rd Line	5	11	0.57	0.064	3	22	0.592	0.094	2	7	1.228	0.453
		5	11	1.294	0.641	3	22	1.316	0.910	2	7	2.785	0.810
F	Rainwater Catchment to 36" 3rd Line	5	11	0.439	0.038	3	22	0.439	0.052	2	7	1.051	0.453
		5	11	1.601	0.641	3	22	1.645	0.725	2	7	2.376	0.590

Figure 49 - Calculated Minor Losses Due to Pipe Fittings for Each Alternative

Total Head Loss		North Line			South Line			Third Line		
	Alternative	Major Head Loss (m)	Minor Head Loss (m)	Total Head Loss (m)	Major Head Loss (m)	Minor Head Loss (m)	Total Head Loss (m)	Major Head Loss (m)	Minor Head Loss (m)	Total Head Loss (m)
50 MGD Demand										
	Current	14.05	0.64	14.69	15.42	0.91	16.33	-	-	-
A	30" Diameter 3rd Line	4.44	0.19	4.63	4.65	0.26	4.91	14.43	0.45	14.88
B	36" Diameter 3rd Line	2.61	0.11	2.72	2.98	0.15	18.20	8.77	0.33	9.10
C	Rainwater Catchment to 30" 3rd Line	2.85	0.64	3.49	0.25	0.02	0.27	8.77	0.09	14.93
		3.51	0.14	3.65	2.98	0.91	18.20	10.55	0.45	11.00
70 MGD Demand										
	30" Diameter 3rd Line	8.97	0.40	9.37	9.68	0.56	10.24	29.16	0.97	30.13
D	36" Diameter 3rd Line	5.76	0.64	6.40	5.68	0.91	6.59	17.48	0.70	30.23
E	Rainwater Catchment to 30" 3rd Line	1.66	0.06	1.72	6.28	0.09	10.24	4.98	0.45	5.43
		5.76	0.64	6.40	8.19	0.91	9.10	17.48	0.81	30.23
F	Rainwater Catchment to 36" 3rd Line	0.99	0.04	1.03	6.28	0.05	10.24	2.88	0.45	3.33
		5.76	0.64	6.40	12.79	0.73	13.52	17.48	0.59	30.23

Figure 50 - Calculated Total Head Loss as the Sum of Major Head Loss and Minor Head Loss

	50 MGD Demand					70 MGD Demand					
	Current	30" Diameter Third Line	36" Diameter Third Line	30" Dia. RW Catchment (Low)	30" Dia. RW Catchment (high)	30" Diameter Third Line	36" Diameter Third Line	30" Dia. RW Catchment(low)	30" Dia. RW Catchment (high)	36" Dia. RW Catchment(low)	36" Dia. RW Catchment (high
North Line											
Length	4535	4535	4535	4535	4535	4535	4535	4535	4535	4535	4535
Diameter (in)	30	30	30	30	30	30	30	30	30	30	30
Diameter (m)	0.76200	0.76200	0.76200	0.76200	0.76200	0.7620	0.7620	0.7620	0.7620	0.7620	0.7620
Flow (gpm)	13040	7065	5360	1658	6050	10285	7809	4147	9392	3150	11621
Flow (m^3/sec)	0.82261	0.44569	0.33813	0.10459	0.38166	0.6488	0.4926	0.2616	0.5925	0.1987	0.7331
velocity (ft/s)	5.91909	3.20693	2.43300	0.75260	2.74620	4.6685	3.5446	1.8824	4.2632	1.4298	5.2750
Velocity (m/s)	1.80381	0.97730	0.74145	0.22935	0.83689	1.4227	1.0802	0.5737	1.2992	0.4357	1.6075
Min Diameter (in	21.30840	15.68440	13.66137	7.59809	14.51408	18.9241	16.4896	12.0165	18.0839	10.4729	20.1156
Reynolds Numbe	983338	532767	404194	125029	456227	775585	588872	312723	708245	237539	876332
Pipe Roughness	2.54E-04	2.54E-04	2.54E-04	2.54E-04	2.54E-04	2.54E-04	2.54E-04	2.54E-04	2.54E-04	2.54E-04	2.54E-04
Friction Factor	0.01605	0.01656	0.01687	0.01905	0.01673	0.0162	0.0165	0.0172	0.0163	0.0177	0.0161
Friction Head Lo	15.85847	4.80238	2.81654	0.30419	3.55751	9.9696	5.8319	1.7209	8.3512	1.0186	12.6564
Minor Head Los	0.64096	0.19315	0.11034	0.00900	0.13800	0.4026	0.2287	0.0640	0.3320	0.0380	0.5080
Total Head Loss	16.49943	4.99552	2.92688	0.31319	3.69551	10.3722	6.0606	1.7849	8.6832	1.0566	13.1644
South Line											
Length	4750	4750	4750	4750	4750	4750	4750	4750	4750	4750	4750
Diameter (in)	30	30	30	30	30	30	30	30	30	30	30
Diameter (m)	0.7620	0.7620	0.7620	0.7620	0.7620	0.7620	0.7620	0.7620	0.7620	0.7620	0.7620
Flow (gpm)	13252	7180	5447	1685	6148	10452	7936	4214	9545	3201	11810
Flow (m^3/sec)	0.8360	0.4529	0.3436	0.1063	0.3878	0.6593	0.5006	0.2658	0.6021	0.2019	0.7450
velocity (ft/s)	6.0153	3.2591	2.4725	0.7649	2.7907	4.7443	3.6023	1.9128	4.3326	1.4530	5.3608
Velocity (m/s)	1.8331	0.9932	0.7535	0.2331	0.8504	1.4458	1.0978	0.5829	1.3204	0.4428	1.6337
Min Diameter (in	21.4809	15.8115	13.7718	7.6597	14.6312	19.0771	16.6231	12.1132	18.2306	10.5573	20.2785
Reynolds Numbe	999325	541439	410755	127065	463617	788178	598449	317775	719782	241385	890584
Pipe Roughness	2.54E-04	2.54E-04	2.54E-04	2.54E-04	2.54E-04	2.54E-04	2.54E-04	2.54E-04	2.54E-04	2.54E-04	2.54E-04
Friction Factor	0.0160	0.0165	0.0169	0.0190	0.0167	0.0162	0.0164	0.0172	0.0163	0.0176	0.0161
Friction Head Lo	17.1436	5.1900	3.0430	0.3284	3.8436	10.7758	6.3028	1.8587	9.0270	1.1000	13.6815
Minor Head Los	0.9096	0.1931	0.1103	0.0160	0.1960	0.5613	0.3220	0.0940	0.4640	0.0520	0.7250
Total Head Loss	18.0533	5.3831	3.1533	0.3444	4.0396	11.3372	6.6248	1.9527	9.4910	1.1520	14.4065
Third Line											
Length		4320	4320	4320	4320	4320	4320	4320	4320	4320	4320
Diameter (in)		30	36	30	30	30	30	30	30	30	30
Diameter (m)		0.762	0.914	0.762	0.762	0.762	0.762	0.762	0.762	0.762	0.762
Flow (gpm)		15120	18536	6790	12945	22000	27000	8870	20095	10880	24650
Flow (m^3/sec)		0.954	1.169	0.428	0.817	1.388	1.703	0.560	1.268	0.686	1.555
velocity (ft/s)		6.863	5.843	3.082	5.876	9.986	12.256	4.026	9.121	4.939	11.189
Velocity (m/s)		2.092	1.781	0.939	1.791	3.043	3.735	1.227	2.780	1.505	3.410
Min Diameter (in		22.945	25.405	15.376	21.231	27.677	30.662	17.574	26.452	19.464	29.297
Reynolds Numbe		1140189	1164823	512029	976174	1659006	2036052	668881	1515351	820454	1858840
Pipe Roughness		5.00E-06	5.00E-06	5.00E-06	5.00E-06	5.00E-06	5.00E-06	5.00E-06	5.00E-06	5.00E-06	5.00E-06
Friction Factor		0.012	0.011	0.013	0.012	0.011	0.011	0.013	0.011	0.012	0.011
Friction Head Lo		14.587	8.759	3.353	10.955	29.193	42.702	5.468	24.681	7.956	36.054
Minor Head Los		0.498	0.364	0.102	0.371	1.065	0.769	0.173	0.889	0.127	0.647
Total Head Loss		15.084	9.123	3.455	11.326	30.258	43.471	5.641	25.570	8.083	36.701

Figure 51 - Table of Calculated Values for Third Pipeline Design

Appendix D – Pedro Miguel River sub-basin Rainwater Catchment Design Calculations

$$H_j = \frac{Q_0 v_0 - Q_i v_i - Q_1 v_1 \cos\theta}{0.5g(A_0 + A_i)}$$

		A (30" pipe)	0.456036731
		A (36" pipe)	0.656692893
		Gravitational Constant	9.81
Q_1 =	0.26 m^3/sec to 1.64 m^3/sec	v_1 =	0.57 to 3.596
Q_i =	0.43 to 0.82 (50 MGD flow)(30" pipe)	v_i =	0.943 to 1.798
	0.56 to 1.27 (70 MGD flow)(30" pipe)		1.228 to 2.785
	0.69 to 1.56 (70 MGD flow)(36" pipe)		1.051 to 2.376
Q_0 =	0.82 (50 MGD flow)(30" pipe)	v_0 =	1.798
	1.27 (70 MGD flow)(30" pipe)		2.785
	1.56 (70 MGD flow)(36" pipe)		2.376

	Maximum Rainwater Catchment Flow			Minimum Rainwater Catchment Flow		
θ	50 MGD (30" pipe)	70 MGD (30" pipe)	70 MGD (36" pipe)	50 MGD (30" pipe)	70 MGD (30" pipe)	70 MGD (36" pipe)
90	0.829590808	1.22755936	0.872977331	0.014843241	0.014843241	0.010307806
95	-0.723622517	-0.325653965	-0.205643034	-0.024188307	-0.024188307	-0.016797436
100	-0.897821838	-0.499853286	-0.326614784	-0.028565857	-0.028565857	-0.019837401
105	0.556563967	0.954532518	0.683375358	0.0079822	0.0079822	0.005543195
110	1.5558718	1.953840351	1.37733913	0.033094354	0.033094354	0.02298219
115	0.668417683	1.066386234	0.761051549	0.010793033	0.010793033	0.007495162
120	-0.834364499	-0.436395947	-0.282547188	-0.026971203	-0.026971203	-0.018730002
125	-0.799475337	-0.401506786	-0.258318604	-0.026094454	-0.026094454	-0.018121149
130	0.723100315	1.121068867	0.7990256	0.012167183	0.012167183	0.008449433
135	1.552005429	1.94997398	1.374654151	0.032997194	0.032997194	0.022914718
140	0.499687848	0.897656399	0.643878053	0.006552929	0.006552929	0.004550645
145	-0.926222675	-0.528254124	-0.346337588	-0.029279558	-0.029279558	-0.020333026
150	-0.682858885	-0.284890333	-0.177334956	-0.023163936	-0.023163936	-0.016086066
155	0.881117848	1.279086399	0.908759997	0.016138092	0.016138092	0.011207009
160	1.525036173	1.923004724	1.355925501	0.032319469	0.032319469	0.022444075
165	0.326369999	0.724338551	0.523518436	0.00219753	0.00219753	0.001526062
170	-0.997580859	-0.599612307	-0.395891882	-0.031072756	-0.031072756	-0.021578303
175	-0.550024273	-0.152055721	-0.085088698	-0.019825862	-0.019825862	-0.01376796

Figure 52 - Angle Calculation for Design of the Junction Between the Third Line and the Rainwater Catchment Pipeline